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INCLUDING PRESTRESSED CONCRETE

MARCH 1961 VOL. LVI. NO. 3

FIFTY-SIXTH YEAR OF PUBLICATION

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LEADING CONTENTS

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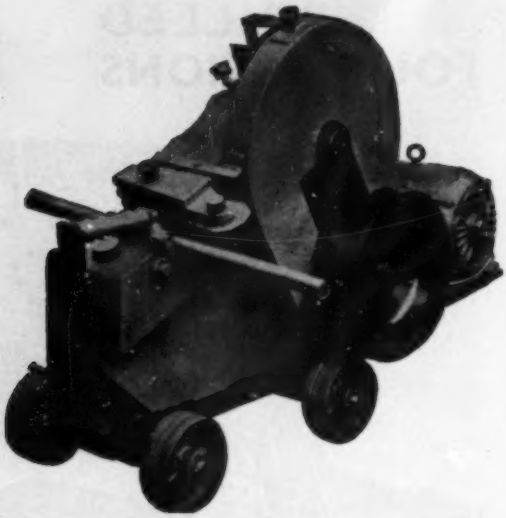

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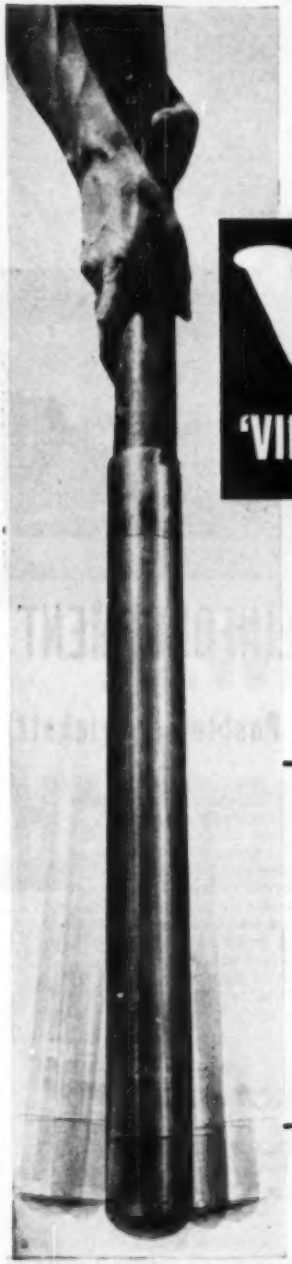

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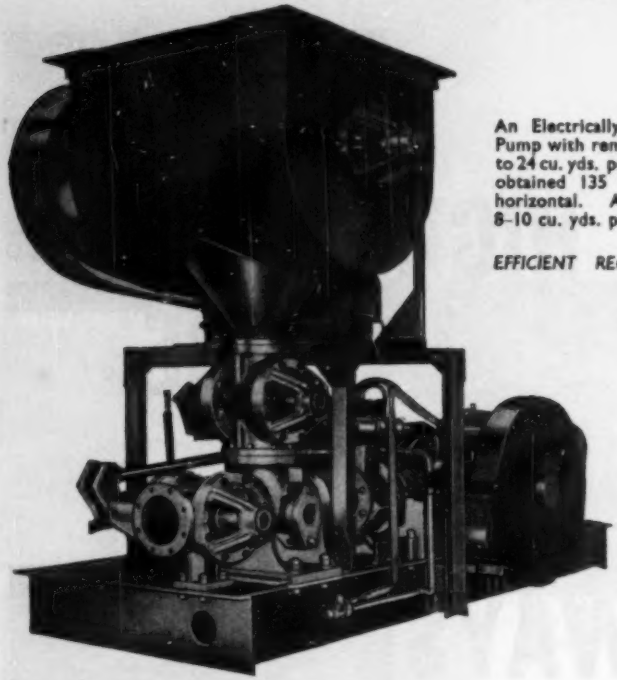

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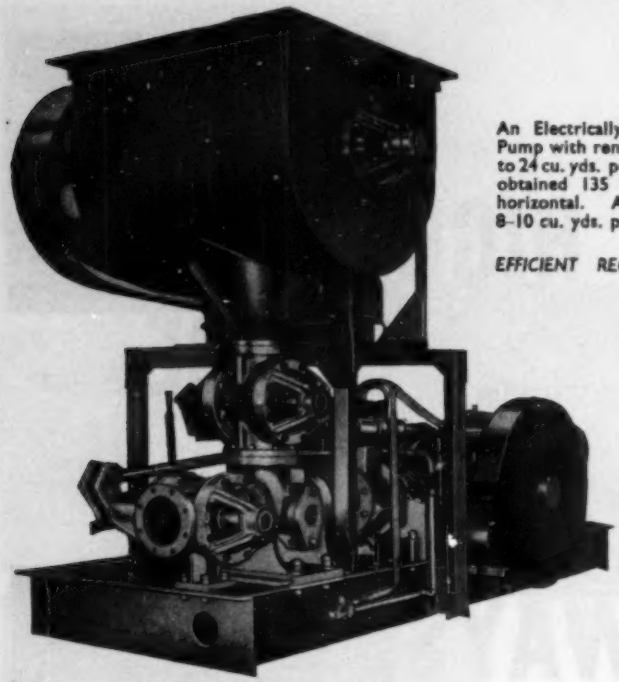

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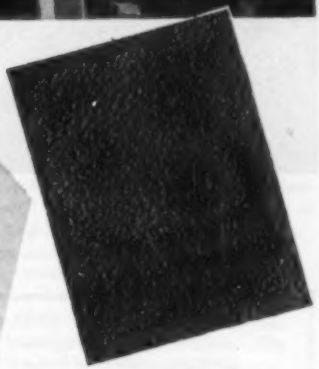

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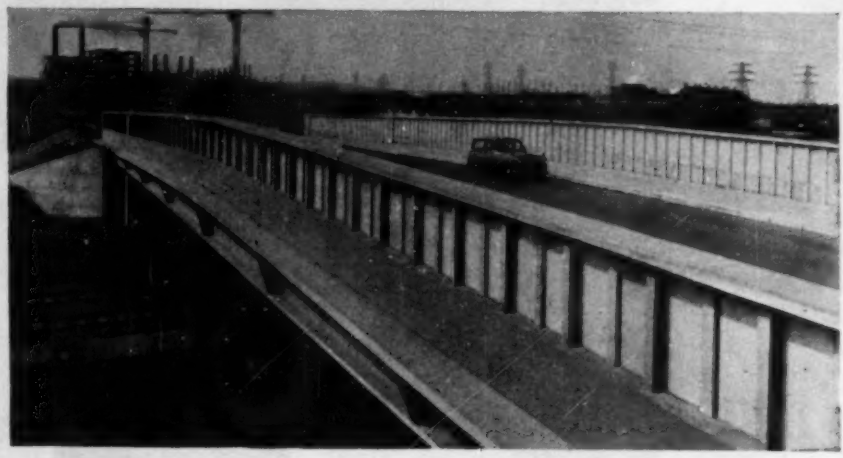
Road bridge reconstruction over railway junction - Renwick Road Bridge, Barking



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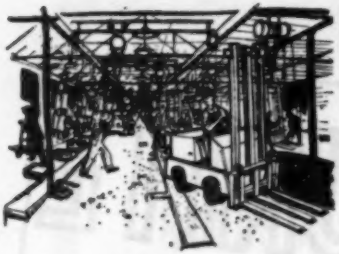

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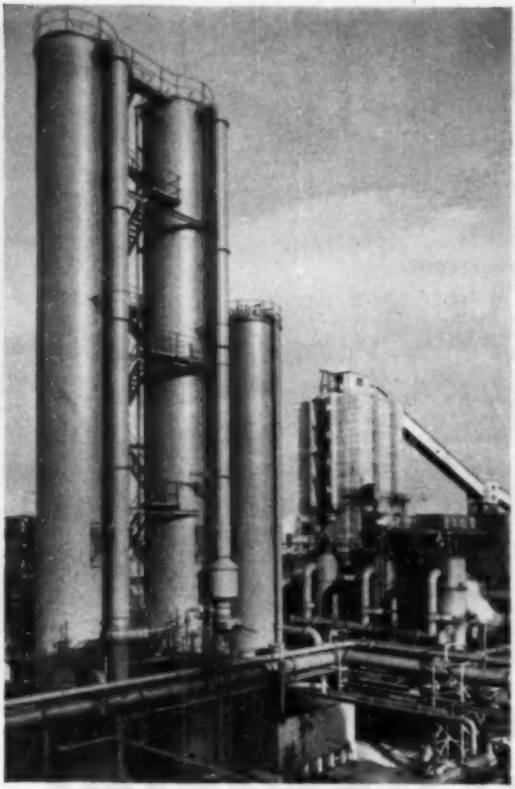

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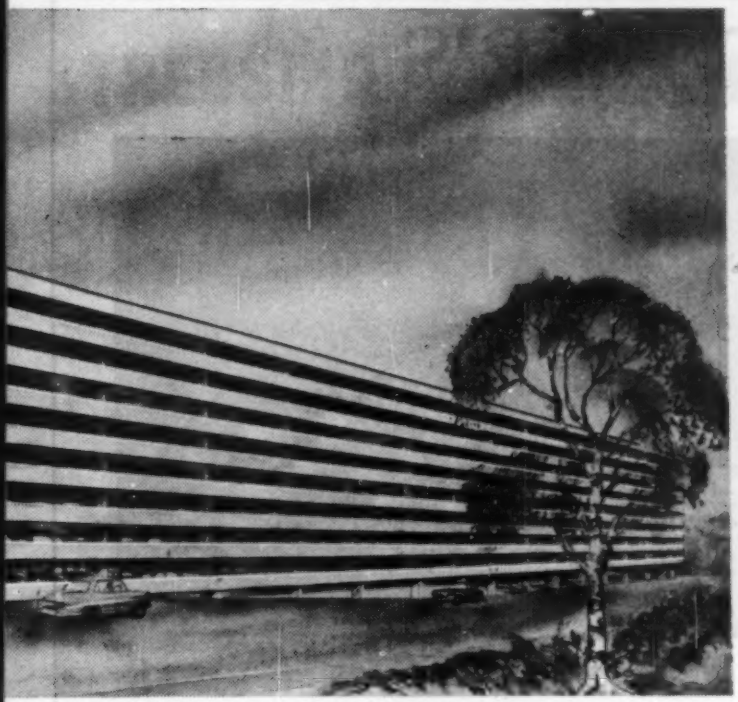

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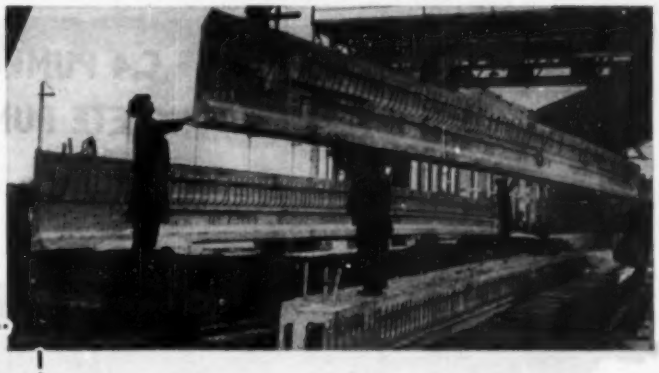

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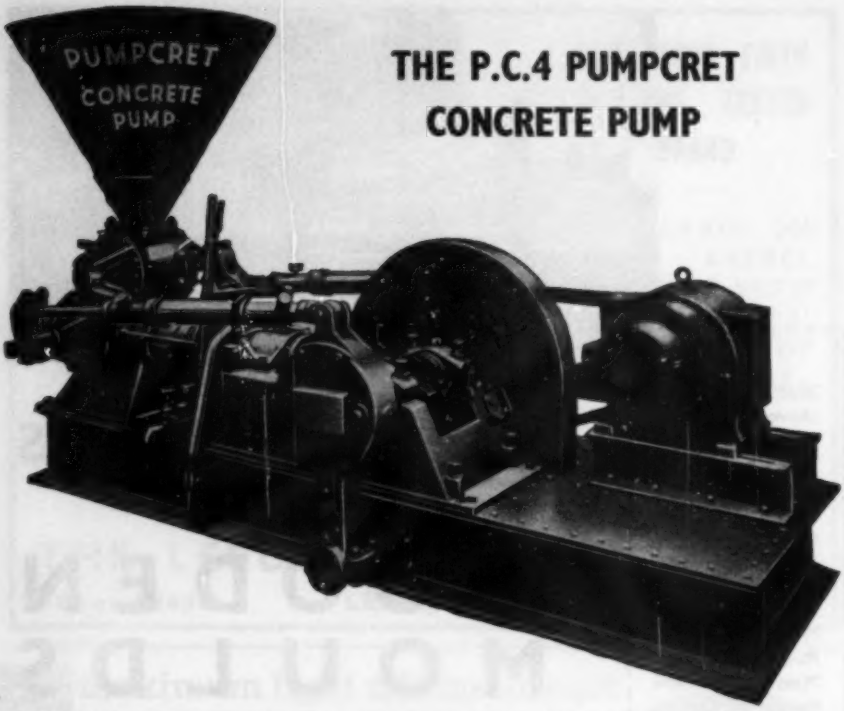

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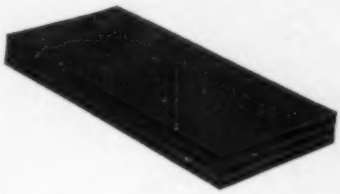

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15	5·90	17
22	8·66	41
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27	10·6	64
30	11·8	81
35	13·8	114
42	16·5	168
48	19·7	245
60	23·6	357
87	34·2	772
92	36·2	866

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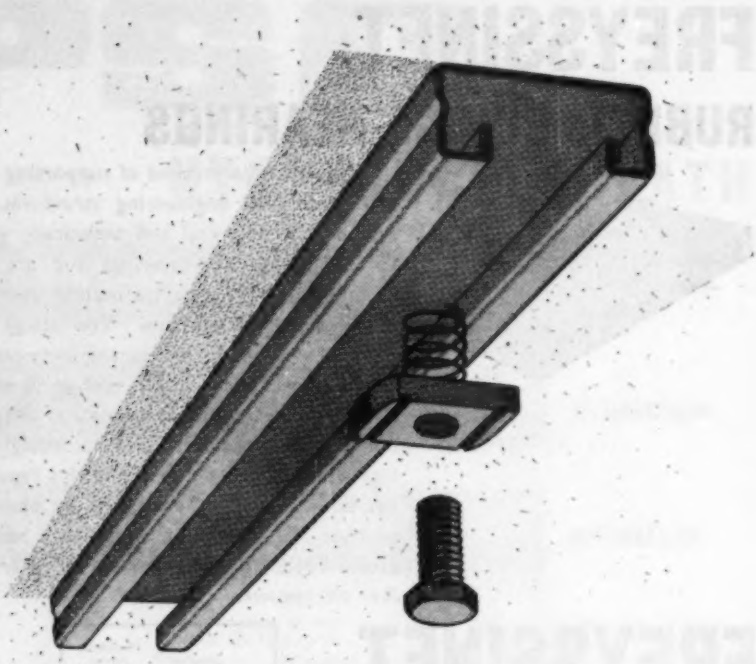

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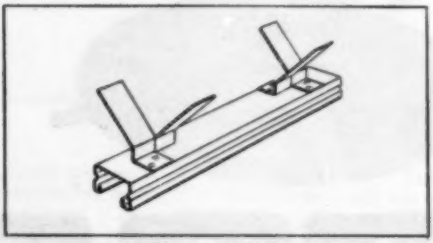

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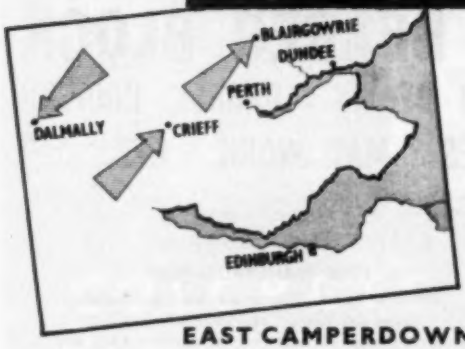

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Volume LVI, No. 3. LONDON, MARCH, 1961.

EDITORIAL NOTES

Statically-indeterminate Structures.

ATTEMPTS to avoid the complexity of rigid analyses of statically-indeterminate structures has led to the production of many methods which are easier to apply and the results of which are of sufficient accuracy for the purpose of structural design. The two principal classes of analyses are those in which the elastic condition at working loads is investigated and those based on the behaviour of the structure as the condition at failure is approached. The latter class include the ultimate-load theories which have gained acceptance only in recent decades and for which the virtues of comparative simplicity and realism are claimed. The former class includes the several classical analyses which were developed during the period dating from a century or more ago.

Many of the elastic theories and methods of analysis fail to take into account the characteristics of the materials of which the structure is to be constructed. The early elasticians, and some of their successors, assume they are dealing with an ideally elastic material and, although steel may conform as near as matters to such a material and prestressed concrete likewise, reinforced concrete definitely does not do so. Most elastic analyses take into account the mis-named moment of inertia (or second-moment of the area) of the cross-section of the structural member, or at least comparative moments of inertia. This geometrical property of a reinforced concrete member is not readily or accurately determinable. It is necessary only to consider such a case as a column, which may be entirely in compression, combined with a tee-beam which has a flange not only of an indefinite width but, if tensile stresses occur in the flange, liable to an indefinite degree of cracking if any. Also the effect on the stresses and deformations of such phenomena as shrinkage, creep, and " built-in " strains are rarely taken into account even if it were possible to do so realistically. The common assumption of a loaded frame being a planar assembly of members ignores the effect of adjacent and less highly loaded members in relieving to some extent the loaded frame. The relation between the elastic theory of structural action and the behaviour of engineering structures was the subject of the James Forrest lecture delivered last month by Professor A. J. S. Pippard at the Institution of Civil Engineers.

From time to time attempts have been made to correlate theoretical with actual behaviour of concrete structures. One of the objects of the Symposium

on the Strength of Concrete Structures held in London in 1956 was to establish such correlation, and one of the principal outcomes of these discussions, and other attempts at correlation, is the support given to ultimate-load or plastic-hinge theories. To-day these theories are being carried a stage further in so far that the practical aspect is being given prominence in the development of methods of analysis. In particular, the efforts being made by the Commission for Hyperstatism of the European Committee on Concrete (Comité Européen de Béton) to correlate theoretical and actual behaviour are noteworthy. Elsewhere in this number, Professor A. L. L. Baker, who is the Chairman of the Commission, gives the basis of a general equation for the ultimate-load analysis of frames taking into account the inelastic deformations. At the meeting of the Committee in Monaco in January last, this equation and a simplified limit equation were, with little modification, accepted.

The evolution of the general equation results from the discussions of the Commission. Previous proposals include an approximate theory, a moment-distribution method, a precise method for continuous beams and a limit method. The general equation is applicable without difficulty to simple frames and continuous beams but other methods, such as a limit method with simplifying assumptions, may be more suitable for frames which are many times statically indeterminate. It is possible that eventually the general equation will be extended to three-dimensional frames.

The application of the general equation depends on the availability of moment-rotation diagrams for common typical sections and distributions of bending moments with or without direct load. Practical tests, the object of which is to obtain this data, are proceeding in several laboratories in different countries as part of the programme of research co-ordinated by the Commission. The more research is directed to this subject, the more comprehensive will be the data acquired. To this end, this journal is pleased to accord with the request of the Chairman of the Commission by announcing the Commission's invitation to suitable laboratories not yet taking part to share in this work.

The programme of tests as organised by the Commission is comprehensive but flexible. Three qualitites of concrete are considered having cylinder crushing strengths of 2850 lb., 4300 lb., and 6000 lb. per square inch respectively; if after preliminary tests it is thought that the strength of the concrete is not an important factor, all tests would be made with concrete of the intermediate strength. Four qualities of reinforcement are to be used, namely, mild steel bars, cold-worked smooth and deformed bars, and ordinary high-tensile bars. The effect of binders in over-reinforced beams and in members in which compressive stresses only occur is also to be investigated. The specimens being tested are to be subjected to bending only, to bending and compression with tension reinforcement and with tension and compression reinforcement, to a central concentrated load, and to loads at the quarter-points. Members having various ratios of span to depth are to be tested but, for the time being, the tests are to be limited to members of rectangular cross-section. The countries participating at present are France, Germany, Great Britain, Italy, Russia and the United States of America. The laboratories in each country are allocated a range of variations; for example, in Great Britain, the tests will be made on the strongest concrete with mild steel reinforcement. The number and range of the variants are most extensive, but

it is expected that early tests with extreme parameters will enable a practical range to be determined and so reduce the amount of investigation.

Such widespread research work can be valuable if the tests made by each laboratory are co-ordinated by a central body, thus avoiding that extensive overlapping which is a lamentable feature of so much research work to-day. Some duplication of investigation may be desirable as results may thereby be confirmed (or otherwise).

Bridge Construction in Steel and Concrete.

SEVERAL of the larger bridges now in the course of construction in this country, or about to be commenced, are primarily steel structures. Notable among such structures are the bridges over the River Thames at Maidenhead, where the steelwork will be exposed, and at Runnymede (near Staines), where the steelwork is to be encased in concrete. Both structures are provided with massive abutments of concrete faced with brick. It is not the intention to revive the controversy which was reported in the national press and elsewhere some months ago regarding the relative merits of steel and concrete superstructures for bridges of these spans (270 ft. in the case of Maidenhead Bridge and 174 ft. in the case of Runnymede Bridge), but it cannot go unremarked that the fact that the steelwork of Runnymede Bridge is to be encased in concrete results in a considerable saving in the amount of steel required mainly because of the stiffening effect of the concrete, and this saving is effected in spite of the extra dead load.

No such controversy can, however, be justified in the case of bridges having spans of several thousand feet such as the road bridges now under construction over the River Tamar (span 1100 ft.) near Plymouth, and over the Firth of Forth (span 3300 ft.) in Scotland, or the bridges proposed to be built over the River Severn (span 3240 ft.) and the River Mersey (span 4500 ft.). For such large spans, steel suspension structures are, at present at least, the only practicable type but, for the tall piers required for such bridges, concrete in the form of reinforced concrete in the case of the main piers 240 ft. high for the Tamar Bridge and prestressed concrete in the case of the side piers extending 150 ft. above high water for the new Forth Bridge is the most suitable material. A description of the design and construction of the piers for these two bridges is given on pages 130 and 105 respectively of this journal.

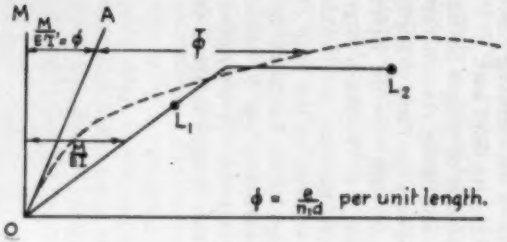


Fig. 1.

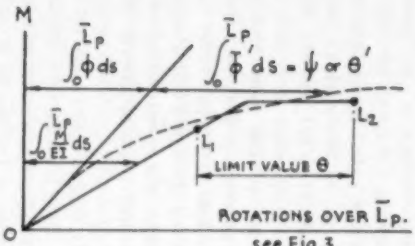


Fig. 2.

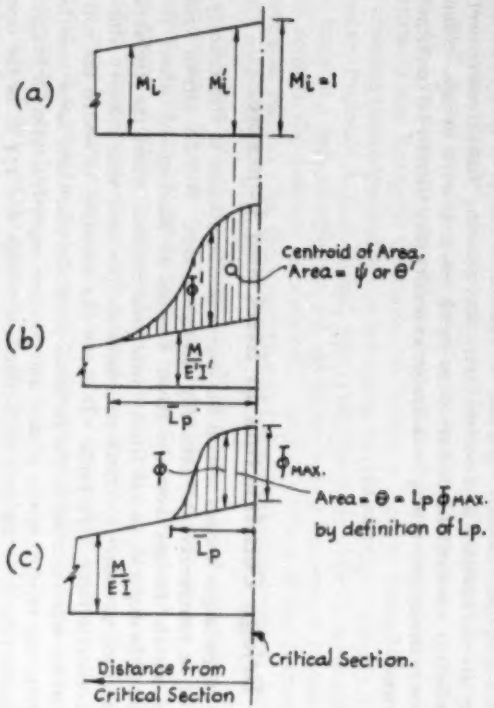


Fig. 3.

A General Equation for Frame Analysis.

(*See facing page.*)

A General Equation for Frame Analysis.

By PROFESSOR A. L. L. BAKER, D.Sc.(Eng.), M.I.C.E., M.I.Struct.E.

THE general equation, which is the subject of this article, is based on inelastic deformations, and has been developed as a result of recent discussions of the Commission on Hyperstatism* in connection with the European Committee for Concrete. Some other proposals put forward to date include an approximate theory[1] and a moment-distribution method[2], both by Y. Guyon, a precise method for continuous beams[3] by G. Macchi, and a limit method[4] by the writer. The general equation may be applied readily to simple frames and continuous beams, but a special application of a modified limit method[4] is probably a more suitable ultimate-load method for more complex frames. The general equation might be extended to three-dimensional frames.[5]

Notation.

Reference should be made to *Figs.* 1, 2, 3 and 4 on pages 96 and 98.

X_i, X_k = Unknown moments (assumed = 1) acting at hinges i and k respectively.

$\bar{X}_i$, $\bar{X}_k$ = Known plastic moments (assumed = 1) acting at hinges i and k respectively.

M = Bending moment generally.

M_i, M_k = Ordinates of diagram of distribution of moments due to $X_i = 1$ and $X_k = 1$ respectively.

M_0 = Ordinate of diagram of moments due to external load.

ϕ_0, ϕ_i, ϕ_k = $\frac{e_{ce}}{n_e d}$ or rotation per unit length due to moment M_0, M_i and M_k respectively.

$\bar{\phi}$, $\bar{\phi}'$ = $\frac{\partial_e}{n_d}$ or inelastic rotations per unit length when $\frac{M}{EI}$ and $\frac{M}{E'I'}$ respectively are assumed to be elastic. (See *Fig.* 3c and b.)

$$\theta_i = \int_0^{\bar{L}_p} \bar{\phi}.ds = \bar{\phi} \text{ max. } L_p. \quad \text{(See } Fig. \text{ 3c at hinge i.)}$$

$$\theta_i' = \int_0^{\bar{L}_p} \bar{\phi}'.ds. \quad \text{(See } Fig. \text{ 3b at assumed hinge section i.}$$

(1). Y. GUYON.—" The Strength of Statically Indeterminate Prestressed Concrete Structures." Symposium on the Strength of Concrete Structures. London. 1956.

(2). Y. GUYON.—" Cadres—Commentaires et Propositions." Note for Commission on Hyperstatism. (Comité Européen du Béton.) 1960.

(3). G. MACCHI.—" Moment Redistribution beyond Elastic Limit and at Failure in Prestressed Concrete Beams." Symposium on Plasticity in Strength of Materials and Structural Design. Varenna. 1956.

" Comments on various Proposals." Note for discussion by Commission on Hyperstatism. (Comité Européen du Béton.) Stockholm. 1960.

Bulletin No. 21. (Comité Européen du Béton.)

(4). A. L. L. BAKER.—" The Ultimate-Load Theory applied to the Design of Reinforced and Prestressed Concrete Frames." Concrete Publications Ltd. London. 1956.

(5). A. L. L. BAKER.—" A General Analysis of Elasto-plastic Three-dimensional Frames." Publications of International Association of Bridge and Structural Engineering. Zürich. 1960.

* Some comments on the work of the Commission are given in this number; see " Editorial Notes ", page 93.

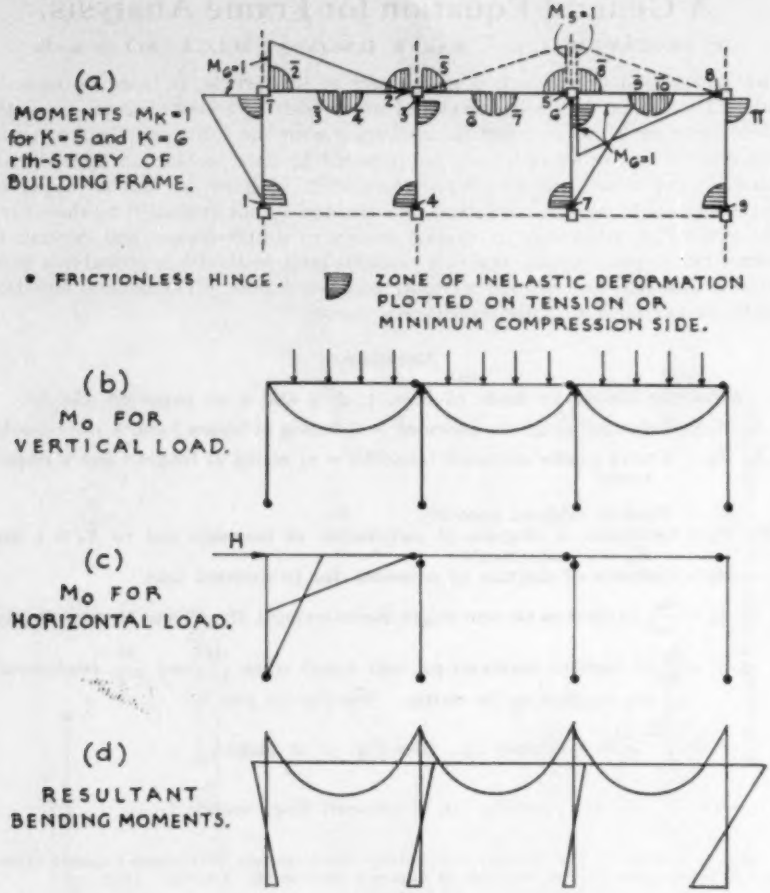


Fig. 4.

ψ = As θ' but at critical section between assumed hinge sections.

$E = \frac{c}{\epsilon_c}$ for values at L_1; $E' = \frac{c}{\epsilon_{cs}}$ for small values of c and ϵ_{cs}.

$I, I' = \frac{Mn_c d}{c}$ for values at L_1; and for small values of c and M respectively.

L_p = Equivalent inelastic length $= \frac{\theta}{\phi \text{ max.}}$. See *Fig.* 3c.

$\bar{L}_p$ = Actual inelastic length. See *Fig.* 3b.

$\epsilon_{cc}, \epsilon_{cu}$ = Strain of concrete at L_1 (or smaller values) and at L_2 respectively.

$\epsilon_{su}, \epsilon_{su}$ = Strain of steel at L_1 and ultimate strain of steel respectively.

$\epsilon_c = \epsilon_{cu} - \epsilon_{cc}$.

c, c_1, c_2 = Concrete stress generally and at L_1 and L_2 respectively.
f_s, f_u = Steel stress at L_1 and ultimate steel stress respectively.
d = Effective depth of section.
$n_1 d$, $n_2 d$ = Neutral-axis depth at L_1 and L_2 respectively.
$\bar{n}d$ = Neutral axis depth for change of strain from L_1 to L_2.
L_1 = A limit point on the moment-curvature diagram of the critical section of a member (see *Fig.* 1) where either the steel or the concrete develops yield stress. Yield stress in the concrete is assumed to be the cylinder strength and to occur at a strain of 0·002, and in the steel when an offset strain of 0·001 develops.
L_2 = a limit point on the moment-curvature diagram, where either the concrete or steel develops ultimate stress. The ultimate stress of concrete is assumed to be cylinder strength and the ultimate strain to be 0·0035 in unbound concrete or 0·01 in well-bound concrete. For steel, the ultimate stress is assumed to be a mean between the yield and ultimate stresses.

At limits L_1 and L_2, the distribution of strain in the concrete across a section is assumed to be linear, and distribution of compressive stress in the concrete to be parabolic.

General Compatibility Equation—Ultimate Condition.

In a frame which is n-times statically indeterminate, insert n-frictionless hinges so that the frame becomes statically determinate and remains stable. Assume external equal and opposite bending moments X_i, X_k, etc., to act at hinge sections i, k, etc., on the members on either side of the hinge sections. Plot moment-diagrams for $X_i = 1$, $X_k = 1$ on the tension, or minimum-compression side of the members. Also, indicate at all critical sections 1 to n and $\bar{1}$ to $\bar{n}$ the zones of inelastic deformation (*Fig.* 4a) on the tension or minimum compression side of the member. At this stage, the sign of the resultant bending moment at the critical sections must be assumed. Checking and adjustment, if necessary, is made later.

When, at any section i, all external loads and moments $X_1 \ldots X_n$ act, continuity at the section is retained by including inelastic deformation as distributed. As an external moment, therefore, X_i does not rotate, and the virtual work done by $X_i = 0$. The virtual work done by X_i (assumed $= 1$) is zero, and by the internal equilibrating restraint moments M_i is zero. The total virtual work done by M_i is due to the following.

1.—Rotations ϕ_0 caused by external load.

2.—Rotations ϕ_i caused by M_i or $X_i = 1$.

3.—All rotations (such as ϕ_k) caused by all moments (such as $X_k = 1$) which act on the same members as $X_i = 1$.

4.—All inelastic rotations (such as $\bar{\phi}'$) which occur in members acted on by X_i.

The sum of the above items of virtual work is zero, which establishes the following general compatibility equation, which is typical for any hinge section such as i.

$$\int M_i \phi_0 . ds + X_i \int M_i \phi_i . ds + \Sigma X_k \int M_i \phi_k . ds + \Sigma \int M_i \bar{\phi}' . ds = 0 \quad . \quad (1)$$

By definition $\phi = \dfrac{e_{cs}}{n_s d} = \dfrac{c}{E' n_s d} = \dfrac{M}{E'I'}$ for small values of e_{cs}, that is $E'I'$ is based on the line OA (*Fig.* 2).

Referring to *Fig.* 3, at any inelastic zone

$$\int M_i \bar{\phi}' . ds = M_i' \int \bar{\phi}' . ds = M_i' \psi = M_i \psi$$

very approximately, since the influence of inelastic rotation is governed by the large deformations which occur close to the critical section. Equation (1) can therefore be written

$$\int \frac{M_i M_0}{E'I'}ds + X_i\int \frac{M_i M_i}{E'I'}ds + \Sigma X_k\int \frac{M_i M_k}{E'I'}ds + M_i\phi_i' + \Sigma M_i\psi_{\bar{n}}^{-} = 0 \qquad (2)$$

For $i = 6$ (refer to *Fig.* 1), equation (2) becomes

$$\int \frac{M_6 M_0}{E'I'}ds + X_6\int \frac{M_6 M_6}{E'I'}ds + \Sigma X_k\int \frac{M_6 M_k}{E'I'}ds + 1.\theta_6' + 1.\psi_{\bar{1}} + 1.\psi_{\bar{2}} - \tfrac{1}{2}.\psi_{\bar{3}} - \tfrac{1}{2}.\psi_{\bar{4}} - 1.\psi_{\bar{5}} + \tfrac{1}{2}.\psi_{\bar{5}} + \tfrac{1}{2}\psi_{\bar{10}} = 0. \qquad (3)$$

The signs are positive when M_i and M_k or M_i and inelastic diagrams are on the same side of a member, and negative when on opposite sides.

To obtain the ultimate strength of a frame or the moment distribution at any stage of loading, the procedure is as follows.

1.—The $\frac{M}{\psi}$ curve (refer to *Fig.* 2) must be known for each critical section.

2.—The X-values must be adjusted until all the ψ-values and their signs are compatible with the resultant moments at each critical section 1 to n and $\bar{1}$ to $\bar{n}$.

This procedure may be practical in the case of simple indeterminate structures, such as continuous beams, when the $\frac{M}{\psi}$ curves are known. In general, however, compatibility is very easily obtained by making certain simplifying assumptions justified by the use of safe limiting values.

Simplified Limit Design.

Assume the $\frac{M}{\phi}$ curve (*Fig.* 2) and $\frac{M}{\psi}$ curve (*Fig.* 2) is linear from 0 to L_1, and that M is constant between L_1 and L_2, except for a small increase of deformation beyond L_1. Then in equation (1), for values at L_1 $\phi = \frac{e_{co}}{n_e d} = \frac{c}{En_e d}$. By definition, $I = \frac{Mn_e d}{c}$ for values at L_1; therefore

$$\int M_i\phi_k.ds = \int \frac{M_i M_k}{EI}ds.$$

If all members between sections 1 to n are designed so that limit L_1 is not exceeded (that is at all sections $\bar{1}$ to $\bar{n}$), then in equation (1) all ψ-terms become zero and X-terms become $\bar{X}$, giving the following typical general equation of compatibility.

$$\int \frac{M_i M_0}{EI}ds + \bar{X}_i\int \frac{M_i M_k}{EI}ds + \Sigma\bar{X}_k\int \frac{M_i M_k}{EI} = -\theta_i \qquad (4)$$

Compatibility of the resultant moment and θ-values is then easily obtained by

1.—Calculating M_0-values from the external load.

2.—Assuming $\bar{X}$-values so that all θ-values are of the correct sign, that is positive when the corresponding $\bar{X}$-value is positive, and so that all θ-values lie between L_1 and L_2 (*Fig.* 4) for the particular critical section.

3.—Designing sections $\bar{1}$ to $\bar{n}$ so that stresses L_1 are not exceeded.

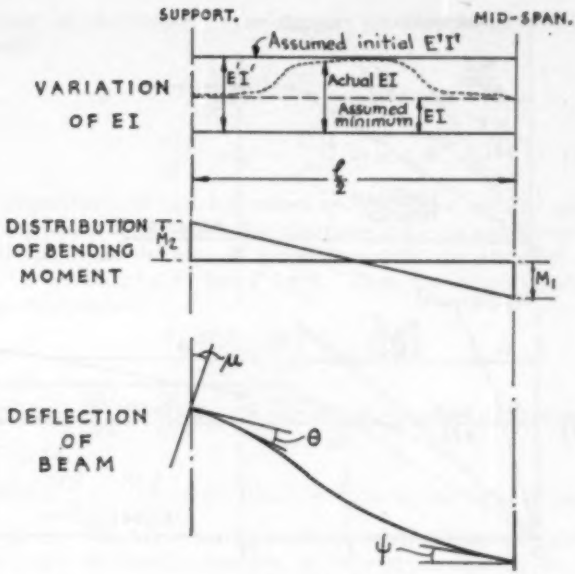


Fig. 5.

To satisfy conditions 1 to 3, the positions of hinges 1 to n are chosen at the probable sections of maximum stress for elastic conditions throughout. $\bar{X}$-values assumed equal to X-values for elastic conditions would then give all θ-values equal to zero. $\bar{X}$-values may be assumed between 0 and elastic X-values, so that the θ-values are not excessive. If hinge sections are designed to have ultimate moments of resistance, that is $\bar{X}$-values slightly less than the elastic $\bar{X}$-values, then θ-values will be positive and probably lie between L_1 and L_2 sections $\bar{1}$ to $\bar{n}$ being designed so that stresses do not exceed values at L_1, that is the frame is designed not to yield between plastic hinges and hinge positions are selected, and the sections at hinges are designed, so that θ-values have the correct sign and are not excessive in value. The process of obtaining compatibility, therefore, merely entails adding the terms of the left-hand side of the equations, and occasionally by design, adjusting $\bar{X}$-values or permissible values of θ at the hinges, so that actual values of θ are not excessive. In each compatibility equation there is only one unknown, namely θ, and the procedure for designing a frame many times statically interdeterminate can therefore be simple.

Calculation of Permissible Values of θ.

Compatibility of the resultant bending moments at plastic-hinge sections with θ-values is satisfied if θ does not exceed a maximum permissible value equal to the increase of rotation which occurs between L_1 and L_2. The following

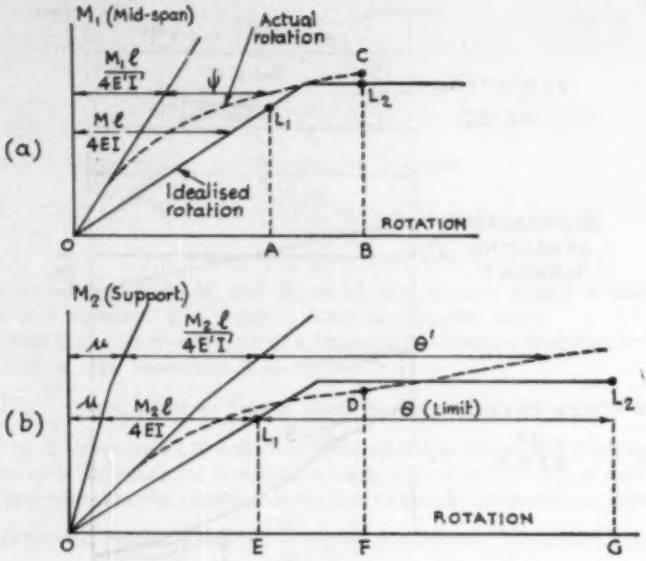


Fig. 6.

semi-empirical formulæ for permissible values of θ are proposed (pending the results of further tests).

$\theta = \dfrac{(e_{cu} - e_{ce})}{n_u} \dfrac{k_1 k_3}{k_2}$ (tension occurs; values at critical section).

$\theta = (e_{cu} - e_{ce}) \dfrac{k_1 k_3}{k_2}$ (no tension occurs; values at critical section). $L_p = \dfrac{k_1 k_3}{k_2} d$.

$k_1 = 1$ for mild steel, $k_1 = 1{\cdot}3$ for cold-worked steel.

k_2 has a linear variation between the following limits (see *Fig.* 3).—$k_2 = 0{\cdot}6$ when $e_g = 3 \times 10^{-5}$ and $k_2 = 2{\cdot}5$ when $e_g = 10 \times 10^{-5}$; $e_g = \dfrac{0{\cdot}002 - e_1}{Z}$, where e_1 is the ultimate strain due to axial load at the section of no bending moment and Z is the distance (in inches) from the critical section to the section of no bending moment. If Z is measured in centimetres, the limiting value of e_g must be multiplied by 0·4. For bending only, $e_1 = 0$; $e_1 = 0{\cdot}002\sqrt{\dfrac{c_1}{0{\cdot}85 c_u}}$, where c_1 is the concrete stress at the point of contraflexure due to an axial force.

$k_3 = 1{\cdot}5$ for concrete having a cube-strength of 2000 lb. per square inch; $k_3 = 1{\cdot}0$ for 6000 lb. per square inch.

Degree of Error in Limit Design.

Consider a typical beam in a building frame (*Fig.* 5). By limit calculations,

a hinge occurs at the support. The support joint rotates μ, and the mid-span slope is zero.

Then
$$\mu + \theta' + \frac{M_2 l}{2E'I'} - \frac{(M_1 + M_2)l}{4E'I'} - \psi = 0. \quad . \quad . \quad . \quad (5)$$

$$\frac{M_1 l}{4E'I'} + \psi = \frac{M_2 l}{4E'I'} + \mu + \theta'. \quad . \quad . \quad . \quad . \quad (6)$$

For compatibility of moment-values and rotations, for any particular load, corresponding points in the broken-line diagrams in *Fig.* 6a and b must be vertically in line. If the beam deforms in accordance with the idealized assumptions, $\psi = 0$, $E'I'$ is replaced by EI and θ' by θ. Then, for compatibility of moment-values and deformation,

$$\frac{M_1 l}{4EI} = \frac{M_2 l}{4EI} + \mu + \theta. \quad . \quad . \quad . \quad . \quad (7)$$

By definition $\frac{M_1 l}{4EI} > \frac{M_1 l}{4E'I'} + \psi$; therefore L_1 is to the right of the broken-line curve in *Fig.* 6a.

Similarly $\frac{M_2 l}{4EI} > \frac{M_2 l}{4E'I'} + \theta'$; therefore L_1 is to the right of the broken-line curve in *Fig.* 6b. By definition, L_2 is always below the dotted curve and to the left of the point of ultimate rotation, or very approximately so.

The case for minimum " actual " ultimate bending strength as compared with " calculated " strength by limit assumptions, occurs when the mid-span critical section designed by limit calculations not to become a plastic hinge, is brittle and the value of μ is such as to cause L_1 (mid-span) to be only just to the right of L_1 (support), that is, so that the rotation of the support hinge by limit calculation is small.

By limit calculation, $M_1 + M_2 = AL_1 + GL_2$.

Actual ultimate value of $M_1 + M_2 = BC + DF$.

It is seen that only in a most extreme case can $AL_1 + GL_2$ exceed $BC + DF$ and never be greater than $AL_1 + EL_1$. If excessively brittle sections are avoided, even when the value of μ is the worst possible, $AL_1 + GL_2$ cannot exceed $BC + DF$.

A similar proposition can be established for a case in which, by limit calculations, a plastic hinge is found to occur at mid-span instead of at the support.

General Comments.

For a rigorous investigation, each possible critical distribution of live-load with corresponding hinges requires investigation. When the live-load is not great in relation to the dead-load, the total distributed load may be assumed to be critical, since then rotations are greatest.

Generally it is sufficient to determine approximately the distribution of bending moments for θ-values = 0. In the case of continuous-beam bridges, it may be important to calculate precisely the influence of inelastic rotations at working load on the distribution of the bending moments.

FIFTY YEARS AGO.

From "Concrete and Constructional Engineering", March, 1911.

A Chimney Constructed with Precast Units.

"The special block used in this method of chimney construction (the Monoshaft method adopted by Messrs. L. G. Mouchel & Partners) has a claw-shaped socket at one end. Each block is 10 in. deep, and of a width and length which can be varied for each shaft, care being taken in the designing to make them of such size that they shall not be unwieldy. The internal part of the claw allows of the passage of the vertical reinforcement, thus securing a good bond; this socket is also of a sufficient width to allow the plain end of the next block to be inserted for a variable distance, thereby making it possible to reduce the diameter of successive courses, and effecting any desired taper to the shaft. In addition to this, the outside of the claws, coming at every external angle, forms an effective vertical rib, and gives a very good appearance to the shaft quite absent from brick chimneys. The space left in the sockets is filled up with a semi-dry concrete similar to the concrete of which the blocks are built, and this leaves the whole one solid mass."

FIG. 1.

The Substructure of the Forth Road Bridge.

THE new road bridge, being constructed for the Forth Road Bridge Joint Board over the Firth of Forth, Scotland, about ½-mile upstream from the famous railway bridge, will be a suspension bridge having a main span of 3300 ft. and two side spans of 1340 ft. each. A clearance of 150 ft. above high water will be provided. The bridge will be the longest suspension bridge in the world outside the U.S.A. A model of the bridge is shown in *Fig.* 1. The deck will be suspended by wire-rope hangers of about 1¾-in. diameter from two main cables which will be about 24 in. in diameter and each will be composed of twelve-thousand 0·196-in. high-tensile steel wires. The cables will pass over two main steel towers more than 500 ft. high and two reinforced concrete side towers (*Fig.* 2) which extend up to the deck. The deck of the main span will be of steel. A reinforced concrete slab on steel girders will be provided for the side spans. There will be two 24-ft. carriageways, two 9-ft. cycle tracks, and two 6-ft. footpaths. The approach viaducts comprise seventeen spans of which the longest will be 170 ft. There will be twelve miles of approach roads.

Anchorage of the Main Cables.

The ends of the main suspension

Fig. 2.—Construction of the Side Towers on the Northern Shore.

Fig. 3.—Cables Protruding from Lower End of Anchorage.

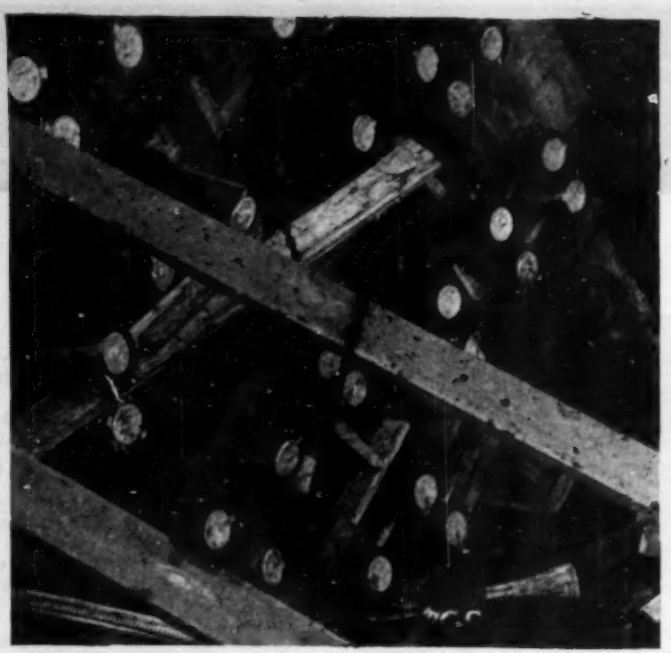


Fig. 4.—Upper End of Anchorage showing Anchors for Prestressing Cables.

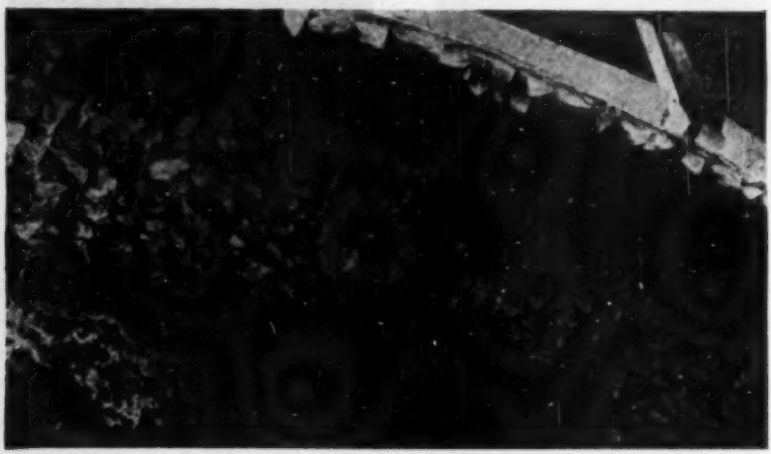


Fig. 5.—Granite Aggregate Stuck to the Inner Face of the Shuttering.

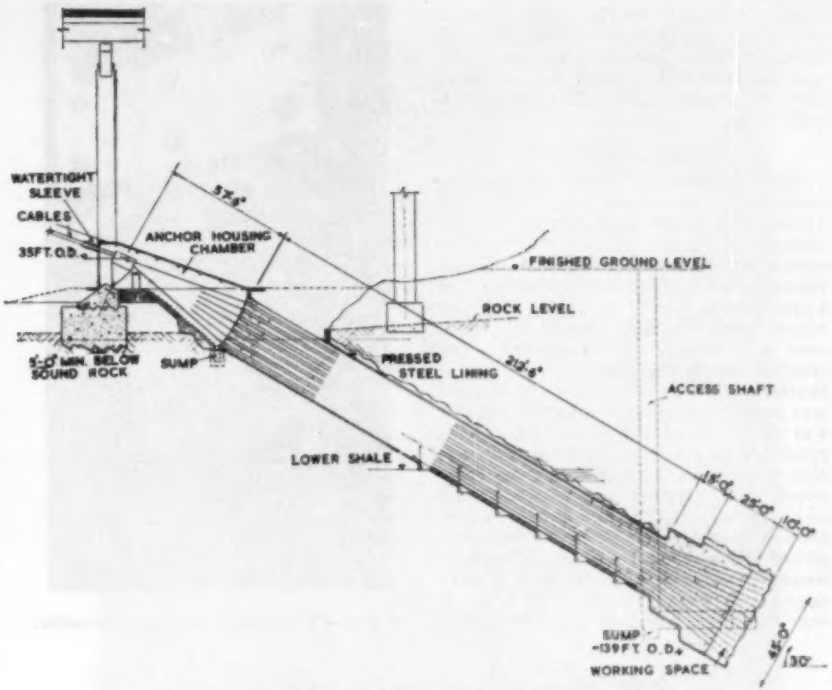


Fig. 6.—Details of Anchor-block on Southern Shore.

cables are secured to four prestressed concrete anchor-blocks cast in tunnels in the rock of the shores of the Firth. The blocks in the southern shore (*Fig.* 6) are about 260 ft. long, from 25 ft. to 45 ft. in diameter and at an angle of 30 deg. with the horizontal. In the northern shore the blocks are shorter, the rock being stronger. Each anchor-block is prestressed with 456 wire strands each of 1¼-in. diameter. The strands are enclosed in metal sheaths, four strands extending from end to end of the anchor-block in each sheath. The upper ends of the four strands in each group are embedded in a white-metal cone which, when the strands are tensioned, is drawn into a steel cylinder bearing on a thrust-plate at the face of the anchor-block (*Fig.* 4). In front of the thrust-plates and tied back to them are steel saddles around which pass the thirty-seven strands of each main suspension cable. Thus the tension in the suspension cable is transferred through the prestressing cables to the lower end of the anchor-block (*Fig.* 3). The strands in each prestressing cable were tensioned at the same time by four hydraulic jacks at the lower end of the anchor-block, and were anchored at that end separately in wedge anchors bearing on steel plates at the face of the concrete.

Each anchor-block was constructed as follows. The tunnel was excavated and the roof supported by pressed-steel lining. The cable sheaths were then placed in position and supported by templates (*Fig.* 7) and the tunnel was filled with concrete except for a working space at the lower end (*Fig.* 6) where the prestressing cables were to be tensioned. When the concrete had hardened the space between the roof of the tunnel and the steel lining was filled with grout under pressure. The cables were then passed into the sheaths and tensioned. Access to the lower ends of the anchor-blocks was by a vertical shaft and horizontal galleries in each

bank. The shaft on the southern bank was about 160 ft. deep. When the prestressing cables had been tensioned the working spaces were filled with concrete and the access shafts and tunnels with rock.

Side Towers.

The side towers are hollow rectangular columns of prestressed concrete. The prestressing cables are made up of wire arranged in parallel and are fixed into the rock at the lower ends and tensioned from the top. Each of the eighty cables applies a prestressing force of 120 tons. Climbing cranes inside the hollow columns (*Fig.* 8) were used to raise the materials for the construction of the columns. One lift of shuttering (*Fig.* 8) about 6 ft. 9 in. high was provided for each concrete tower and was raised by means of hydraulic jacks operating from a steel structure inside the hollow tower. The shuttering was composed of steel frames with wooden facing and was supported on the hardened concrete of the tower while new concrete was placed. For a few feet above ground all concrete is faced with exposed granite aggregate. The pieces of stone (*Fig.* 5) were stuck to the surface of the shuttering

Fig. 7.—Templates for Cable Sheaths.

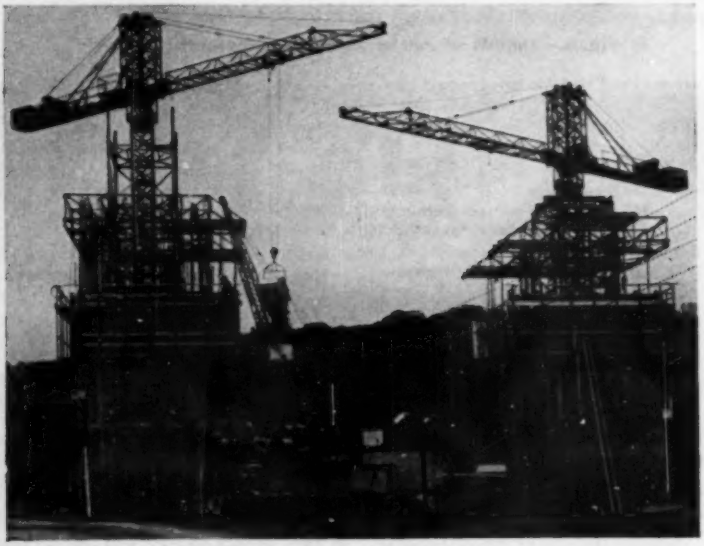


Fig. 8.—Shuttering and Climbing Cranes for the Side Towers.

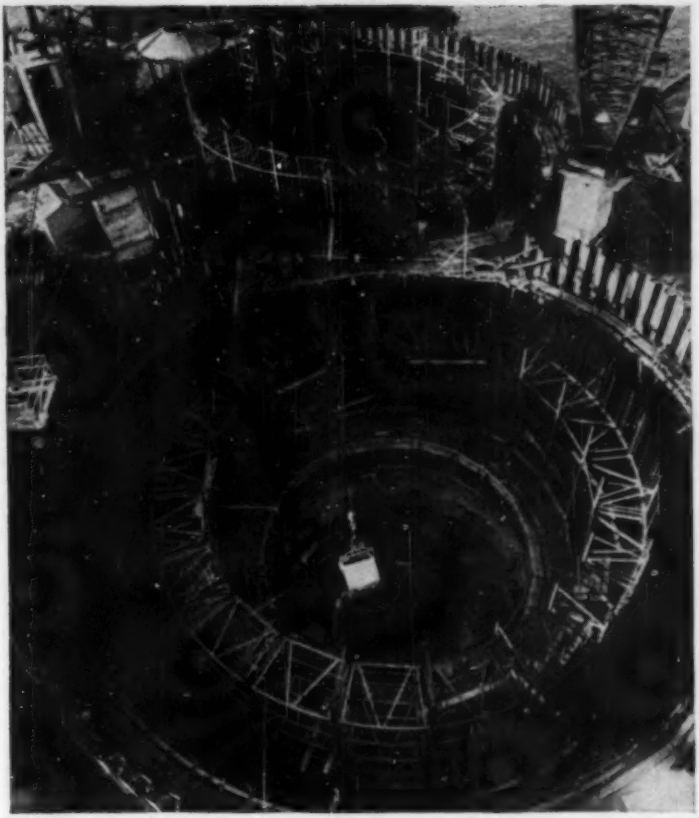


Fig. 9.—Construction of the Main Tower Foundations in Cofferdams.

with a water-soluble adhesive immediately before placing the concrete. All other concrete surfaces were treated with a white cement finish.

Foundations.

The foundation of the southern main tower was constructed in double circular open cofferdams (*Fig.* 9). Two steel and concrete cylinders were sunk through boulder clay to rock which was 100 ft. below water. The concrete foundation pier was cast on the cylinders. The steel bases of the main towers are embedded in the foundation piers, to which the vertical thrust is transferred by a number of steel lugs projecting from, and welded to the side of, the base and transmitting the load to the foundation over a depth of several feet.

Concrete for the southern anchor-blocks was prepared in the plant shown in *Fig.* 10. The mixer is a 1-cu. yd. high-speed machine driven by a 35-h.p. electric motor. Concrete was pumped from the mixer to the anchor-blocks by means of air compressed in the cylinder on the right of the illustration. The cement for the works on the southern side is being brought in special railway trains of 20-ton pressurised-wagons from which the cement is discharged pneumatically at the rate of

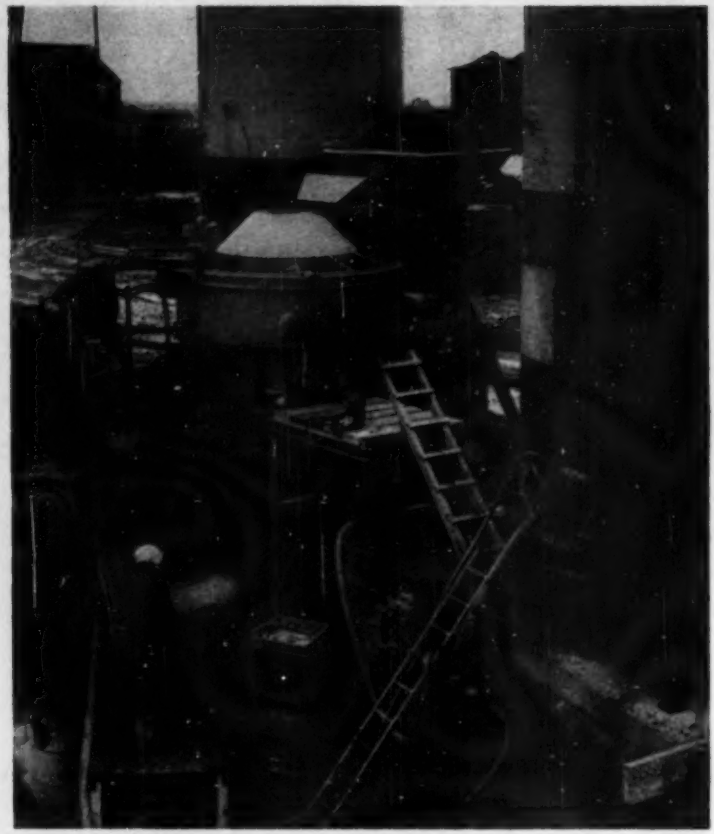


Fig. 10.—Concrete Mixing and Pumping Plant for the Southern Anchorages.

a ton per minute by a pump operated by a diesel motor. The cement is fed into pressurised-tank road vehicles in which it is transported to the site. Up to 500 tons of cement is delivered by the train to South Leith where there is a 200-ton silo which is filled directly from the rail-wagons and from which extra supplies are taken when required on the site.

The consulting engineers are Messrs. Mott, Hay & Anderson. The contractors are Messrs. John Howard & Co., Ltd.

Publications Received.

"**Hydraulics Research 1959.**" Report of the Hydraulics Board of the Department of Scientific and Industrial Research. (London: H.M.S.O. 1960. Price 5*s.* 6*d.*)

"**Basic Road Statistics.**" (London: British Road Federation. 1960. Price 1*s.*)

"**A Guide to Engineers on the Making Up of Private Streets.**" Department of Scientific and Industrial Research. Road Note No. 26. (London: H.M.S.O. 1960. Price 1*s.*)

The Design of Eccentrically-loaded Circular Columns by the Load-factor Method.—II.*

By J. D. BENNETT, B.Eng.

Columns with Cold-worked Reinforcement.

The difference between mild steel and cold-worked steel, apart from the different permissible stresses, is that cold-worked steel is not fully elastic up to the guaranteed yield stress. To determine the secant modulus E_s of cold-worked steel at any particular stress, the simplified stress-strain relationship is shown in *Fig.* 1, from which it is seen that the steel is assumed to be fully elastic up to a

(*Continued on page 117*)

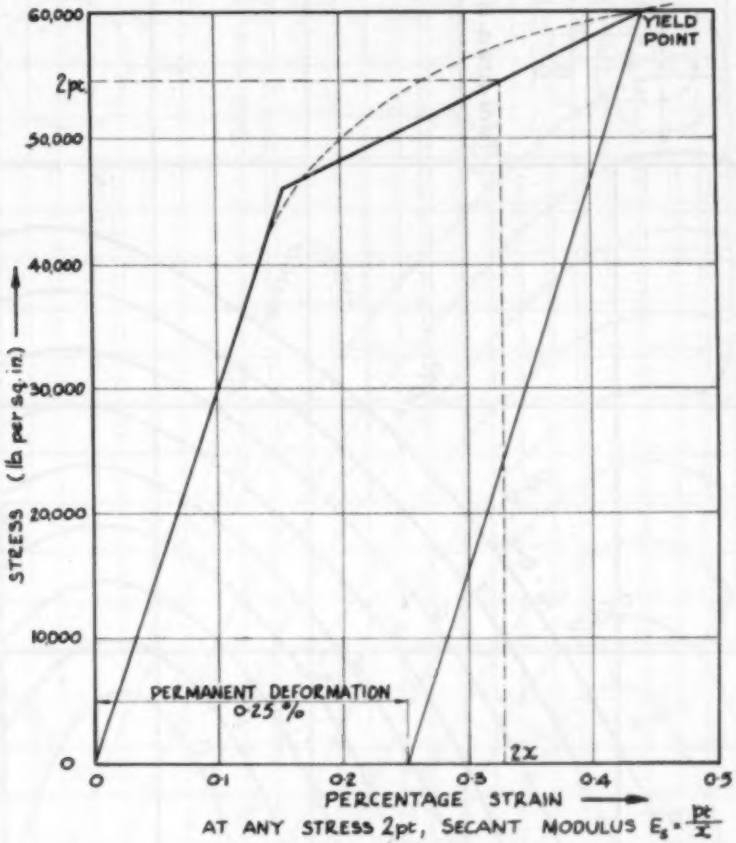
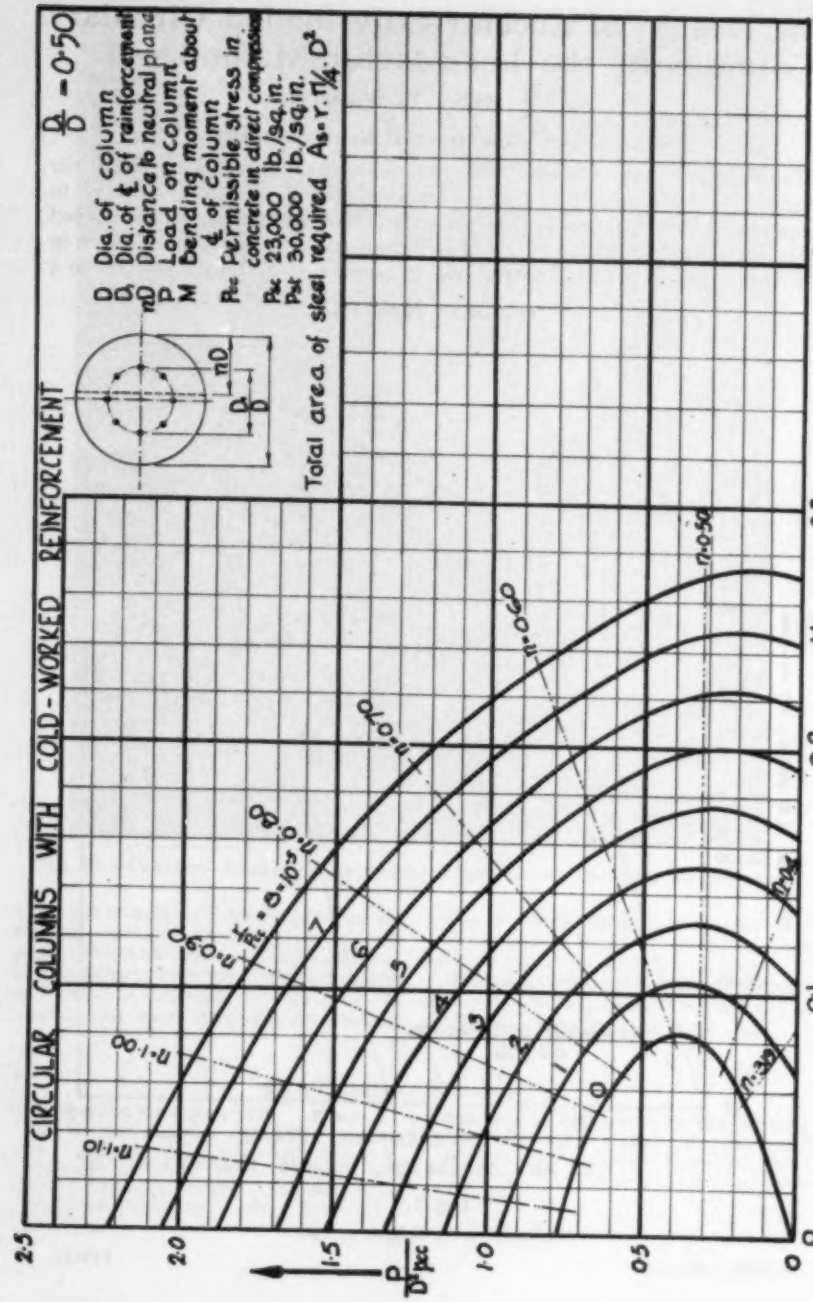
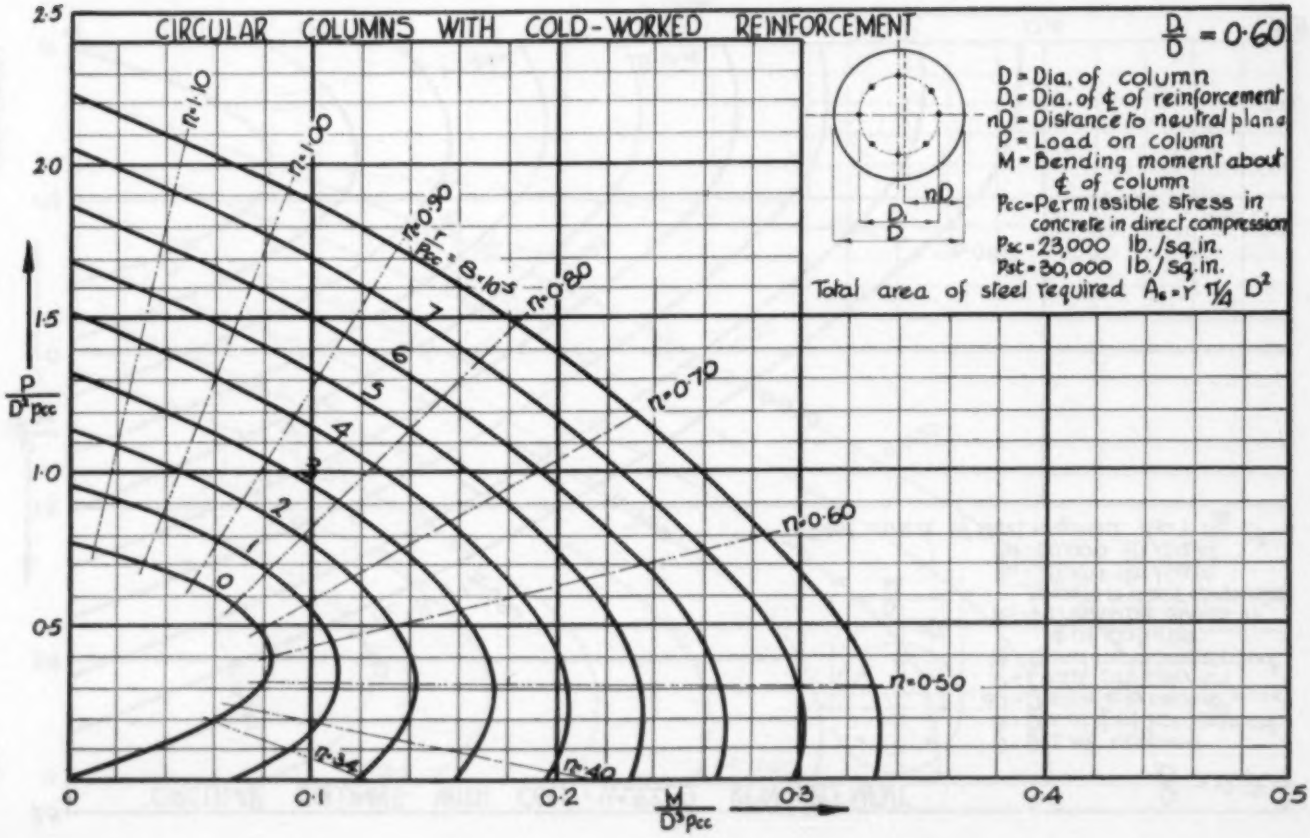
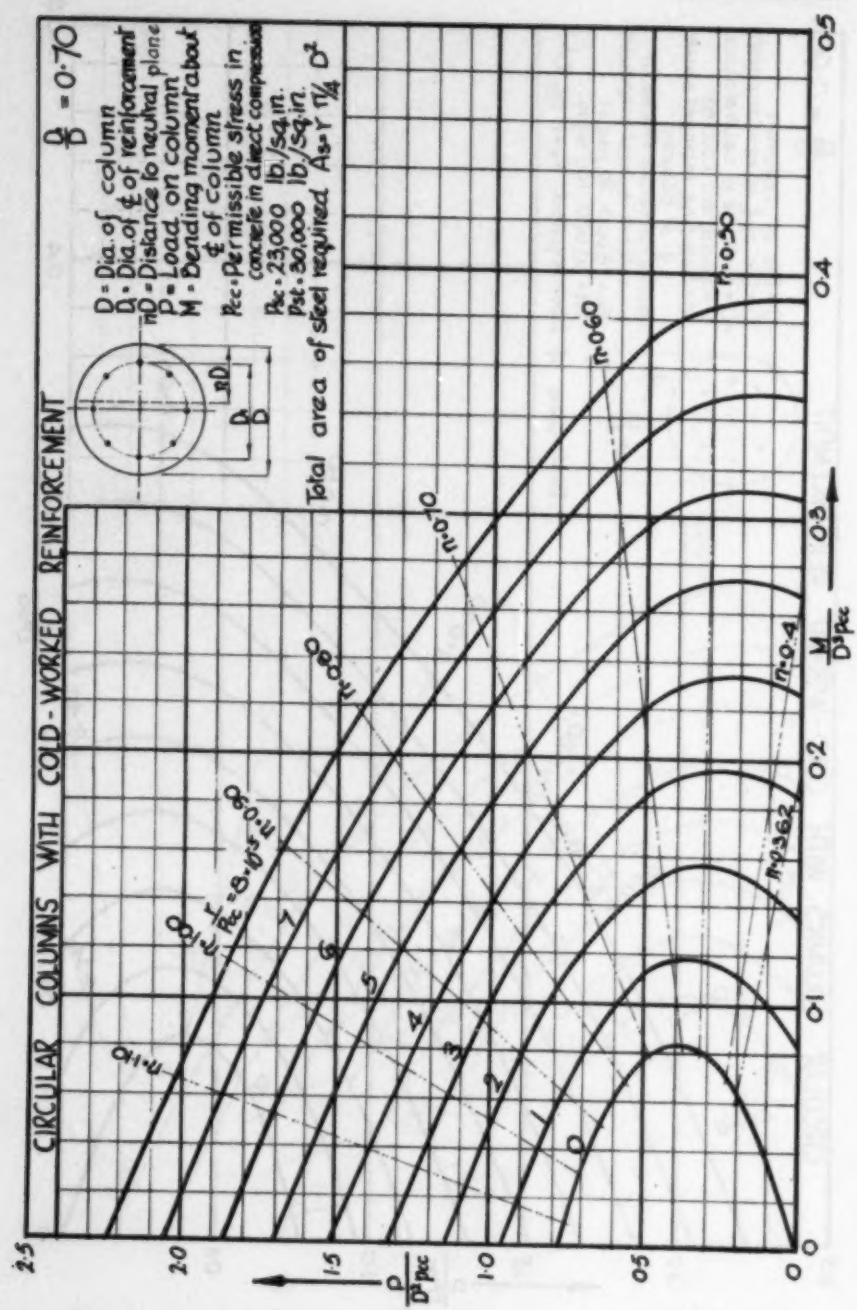


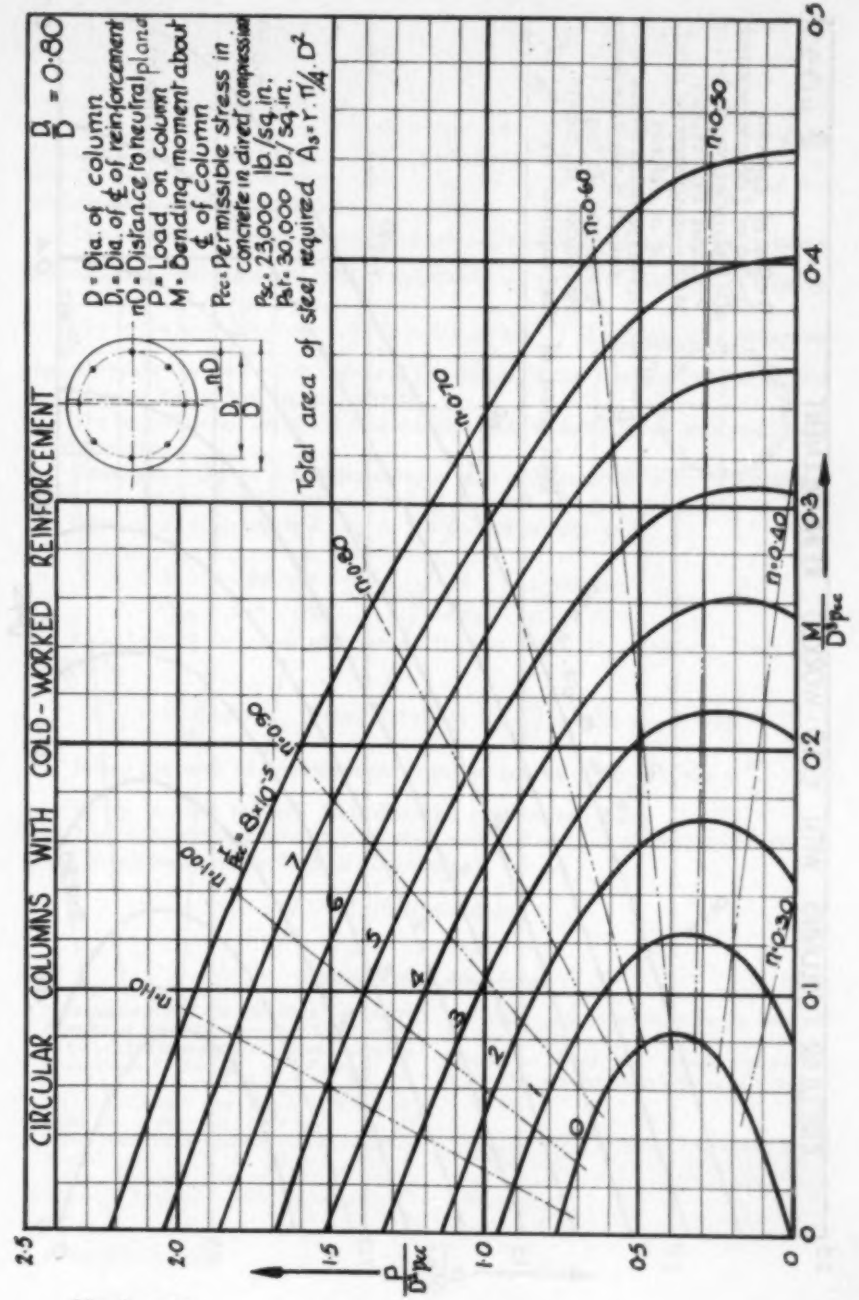
Fig. 1.

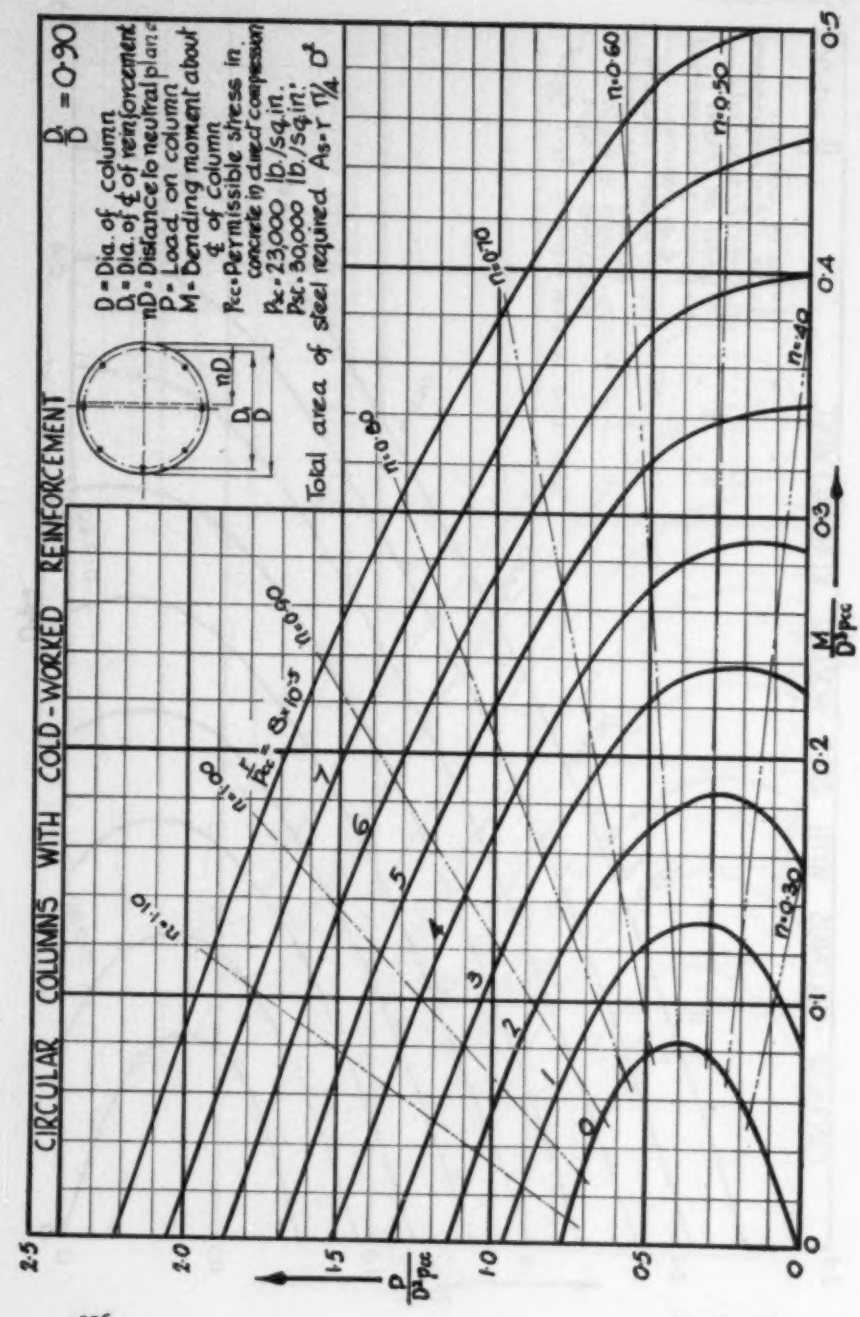
* Continued from February, 1961.


CIRCULAR COLUMNS WITH COLD-WORKED REINFORCEMENT
D/D = 0·50
D Dia. of column
D1 Dia. of ℄ of reinforcement
nD Distance to neutral plane
P Load on column
M Bending moment about ℄ of column
Pcc Permissible stress in concrete in direct compression
Pbc 23,000 lb./sq.in.
Pst 30,000 lb./sq.in.
Total area of steel required As = r.¼.D²
n=0·050
n=0·060
n=0·070
n=0·080
n=0·090
n=0·100
n=0·110
n=0·120
n=0·130
n=0·140
n=0·150
n=0·160
n=0·170
n=0·180
n=0·190
n=0·200
0
1
2
3
4
5
6
7
8
9
10
0·5
0·4
0·3
0·2
0·1
0
2·5
2·0
1·5
1·0
0·5
0
P/D²Pcc
M/D³Pcc


CIRCULAR COLUMNS WITH COLD-WORKED REINFORCEMENT
D/D = 0·60
D = Dia. of column
D1 = Dia. of ℄ of reinforcement
nD = Distance to neutral plane
P = Load on column
M = Bending moment about ℄ of column
Pcc = Permissible stress in concrete in direct compression
Psc = 23,000 lb./sq.in.
Pst = 30,000 lb./sq.in.
Total area of steel required As = r π/4 D²
nD
D1/D
n=1·10
n=1·00
n=0·90
n=0·80
n=0·70
n=0·60
n=0·50
n=·34
n=·40
0
1
2
3
4
5
6
7
P/D²pcc
M/D³pcc
0
0·1
0·2
0·3
0·4
0·5
0·5
1·0
1·5
2·0
2·5


CIRCULAR COLUMNS WITH COLD-WORKED REINFORCEMENT
d/D = 0·70
D = Dia. of column
d = Dia. of ℄ of reinforcement
nD = Distance to neutral plane
P = Load on column
M = Bending moment about ℄ of column
Rcc = Permissible stress in concrete in direct compression
Rbc = 23,000 lb./sq.in.
Rst = 30,000 lb./sq.in.
Total area of steel required As = r ¼ D²
n = 0·10
n = 0·20
n = 0·30
n = 0·40
n = 0·50
n = 0·60
n = 0·362
Rbc/Rst = 0·767
0
1
2
3
4
5
6
7
P/D²Rbc
M/D³Rbc
0
0·5
1·0
1·5
2·0
2·5
0·1
0·2
0·3
0·4
0·5


CIRCULAR COLUMNS WITH COLD-WORKED REINFORCEMENT
D = Dia. of column
D1 = Dia. of ℄ of reinforcement
nD = Distance to neutral plane
P = Load on column
M = Bending moment about ℄ of column
Pcc = Permissible stress in concrete in direct compression
Psc = 23,000 lb/sq.in.
Pst = 30,000 lb/sq.in.
Total area of steel required As = r.π/4.D²
D1/D = 0·80
Es/Ec = 8×10·5
n·0·30
n·0·40
n·0·50
n·0·60
n·0·70
n·0·80
n·1·00
n·1·10
0
1
2
3
4
5
6
7
P/D²pcc
M/D³pcc
0
0·5
1·0
1·5
2·0
2·5
0·1
0·2
0·3
0·4
0·5


CIRCULAR COLUMNS WITH COLD-WORKED REINFORCEMENT
D' / D = 0·90
D = Dia. of column
D' = Dia. of ℄ of reinforcement
nD = Distance to neutral plane
P = Load on column
M = Bending moment about ℄ of column
Pcc = Permissible stress in concrete in direct compression
Psc = 23,000 lb./sq.in.
Psc = 30,000 lb./sq.in.
Total area of steel required As = π/4 D²
n=0·60
n=0·50
n=0·40
n=0·30
n=0·70
n=0·80
n=0·90
n=1·00
n=1·10
n=1·20
Ac / As = 8×10³
0
1
2
3
5
6
7
0·5
0·4
0·3
0·2
0·1
0
2·5
2·0
1·5
1·0
0·5
0
P / D²pcc
M / D³pcc

stress of 46,000 lb. per square inch with a modulus of elasticity of 30×10^6 lb. per square inch. At stresses greater than 46,000 lb. per square inch a straight-line relationship is assumed between stress and strain as far as the yield stress of 60,000 lb. per square inch, which occurs when the permanent deformation is $\frac{1}{4}$ per cent. and the elastic deformation calculated on a modulus of 30×10^6 lb. per square inch is 0·2 per cent. These assumptions are safe for a steel complying with the minimum requirements of the Code and enable the secant modulus of the steel to be readily calculated for any stress.

The notation and other assumptions are as given in Part I and, as for columns with mild steel reinforcement, five design charts are given for values of $\frac{D_1}{D}$ ranging from 0·5 to 0·9 and values of $\frac{r}{p_{cc}}$ from 0 to 8×10^{-5}. The values of n shown on the charts can be used when checking the design, if the reinforcement is widely or unevenly distributed in the column.

The procedure for using the charts is the same as for columns with mild steel reinforcement.

Example.—Design a circular column with a diameter of 2 ft. to support a direct load of 300,000 lb. and a bending moment of 1,500,000 lb.-in.

Using a 1 : 2 : 4 concrete: $p_{cc} = 760$ lb. per square inch.

Therefore

$$\frac{P}{D^2 p_{cc}} = \frac{300{,}000}{24^2 \times 760} = 0{\cdot}685, \quad \frac{M}{D^3 p_{cc}} = \frac{1{,}500{,}000}{24^3 \times 760} = 0{\cdot}143.$$

Providing $1\frac{1}{2}$ in. cover of concrete to $\frac{1}{4}$-in. links, $D_1 = 24 - 3 - 0{\cdot}5 - 1 = 19{\cdot}5$ in.

$$\frac{D_1}{D} = \frac{19{\cdot}5}{24} = 0{\cdot}81.$$ From the chart for $\frac{D_1}{D} = 0{\cdot}80$, $\frac{r}{p_{cc}} = 2{\cdot}25$.

Hence the area of reinforcement required is $2{\cdot}25 \times 10^{-5} \times 760 \times \frac{\pi}{4} \times 24^2 = 7{\cdot}74$ sq. in., say thirteen $\frac{7}{8}$-in. diameter cold-worked bars.

The basis of the charts for mild steel and cold-worked reinforcement will be given in subsequent numbers of this journal.

(To be concluded).

Publications Received.

* **"Background of the Swedish Tentative Standard Specifications for Limitation of Crack Widths in Reinforced Concrete Structures."** By P. O. Jonsson, J. Osterman and G. Wastlund. (Stockholm: Swedish Cement and Concrete Research Institute. 1960. Price Kr. 2.)

* **"Slip Between Reinforcement and Concrete."** By S. T. A. Ödman. (Stockholm: Swedish Cement and Concrete Research Institute. 1960. Price Kr. 2.)

* These papers are reprints in English of contributions to the RILEM Symposium on Bond and Crack Formation in Reinforced Concrete held in Stockholm in 1957.

"National Civil Engineering Laboratory of Portugal." (Lisbon: Ministry of Public Works. 1960. In Portuguese and English.)

Book Reviews.

"The Technology and Properties of Heavy Concrete." Edited by A. E. Desov. [Moscow: Gosstroiizdat (State Publishing Office for Literature on Building, Architecture and Building Materials). 1959. 10r. 50k. (about 95s.)]

THIS book, in the Russian language, contains eight papers dealing with research on three aspects of heavy concrete, namely, compaction by vibration, shielding from radioactive sources, and autoclaving. The subjects of the papers are: Efficient methods of vibrating concrete. Methods of casting silicate mixtures by vibration. Criteria for uniform intensities of vibrations with different frequencies. Properties of heavy concrete designed for shielding from radioactive sources. The design of special concretes of given density, strength and workability. Some questions on the technology and properties of autoclaved concrete. Factors affecting the strength of autoclaved concrete. Research on the modulus of elasticity of autoclaved concrete. The papers have been compiled after studying relevant American and European literature. Some of the conclusions are as follows:

Tables vibrating in two directions are recommended for casting large pieces. Limonite concrete has good resistance to frost. Reinforcement does not corrode in concrete made without the addition of calcium chloride when irradiated with gamma-rays, but in the presence of calcium chloride corrosion is more intense than when the specimens are not irradiated. Bonding of reinforcement in concrete cured in an autoclave is generally no different from bonding in concrete cured by normal methods.

Steam-treatment of concrete made with high-alumina cement at 174 deg. C. gives strengths at one day three times that normally obtained. Autoclaved concrete with compressive strength of over 7000 lb. per square inch can be obtained from a mixture comprising 24 per cent. of high-strength Portland cement, 16 per cent. of ground sand and 60 per cent. of unground sand, using a water-cement ratio of 0·45. The modulus of elasticity of autoclaved concrete is significantly lower than of normally-cured concrete one to three days after steaming. Increasing the temperature of autoclaving lowers the modulus of elasticity by altering the composition of the hydration products and size of crystal by increasing the flaws and by drying the concrete more fully.

"Bauen mit Stahlbetonfertigteilen." (Building with Prefabricated Reinforced Concrete Units.) By László Mokk. [Budapest: Akadémiai Kiadó (Publishing House of the Hungarian Academy of Sciences.) 1960. 7.00 dollars.]

THIS book is a German translation of the second edition of a Hungarian book which has already been translated into several languages in Eastern Europe. It is a comprehensive text book on the use of precast concrete members in industrial buildings. The trend towards prefabrication in concrete is general in most countries but is probably more widespread in countries behind the Iron Curtain than in the western hemisphere. It was particularly so in Hungary where the shortage of timber and steel after World War II compelled engineers to concentrate their effort on eliminating scaffolding and reducing the amount of shuttering.

The book is an impressive record of the achievement of Hungarian engineers since 1947. It is divided into six sections, the first of which contains a short history of prefabrication in concrete and establishes the basic principles of this method as regards work in factories and at sites. The essential condition for prefabrication is repetition which must be taken into account at the design stage. Since the connection of the precast members is one of the main difficulties, it is advantageous to have the least number of joints. The greatest size of a member is governed by the equipment available for transportation and erection. Owing to the high cost of transporting heavy loads in Hungary, large members are generally precast at the site.

The second section deals with the subdivision of various types of structures into convenient units and with their joints. Many interesting and ingenious solutions are described in great detail and illustrated.

The third section is devoted to problems of manufacture, site organisation, moulds, lifting and erection. It is re-

commended that moving heavy members horizontally should be avoided. Such members should be cast in a position which allows them to be erected by tilting and lifting only. The greatest weight for which lifting equipment is now available in Hungary is 60 tons.

In the fourth section various types of cranes and masts and the methods of using them are described. The need for taking safety precautions for the workmen is emphasised.

The fifth section contains about seventy typical examples of completed structures, mainly in Hungary, but also in other countries including England. In many cases the greatest weights of members and the quantities of concrete and steel (per square metre of area covered) are given. Most impressive is an Hungarian power station with columns up to 108 ft. long and beams up to 137 ft. long, both members weighing 60 tons each. The beams had to be lifted to a height of over 100 ft. In another power station ell-shaped wall units 108 ft. high and 13 ft. wide, with exposed-aggregate surfaces, were used resulting in a novel architectural appearance. There are also many examples of prefabricated shell structures of various shapes.

In the last section the economic aspects of prefabrication are summarised. These vary from country to country, depending on the relative costs of labour and materials, and, where timber is scarce, on the availability of foreign currency.

The book is very well illustrated with diagrams and photographs many of which are self-explanatory so that it should be of great interest even to readers without a knowledge of German.—K. H.–K.

Tests on Joints in Precast Members.

DR. K. HAJNAL-KÓNYI writes as follows.

" Some of the results given in the article entitled ' Tests on Joints subjected to Bending and Shearing in Precast Members ' by W. J. Larnach, which was published in this journal for December 1960, are rather puzzling.

For instance, the ' plain beams ' corresponding to joints Nos. 1, 4, 5 and 6 were each reinforced with two plain ¾-in. mild steel bars. Although according to B.S. Code No. 114 these beams were ' over-reinforced ' they were, in fact, under-reinforced and failure should have occurred by yielding of the steel. The variation of the cube-strength was comparatively small. With an assumed cylinder-strength of 4000 lb. per square inch and an assumed yield-point stress of the steel of 36,000 lb. per square inch, I calculate that the ultimate moment of resistance is approximately 60 in.-ton, which agrees with the results of beams Nos. 4 and 5, since the yield-point stress was probably higher than assumed by me. How, however, can the resistances of the first and last beams (43·5 and 75·0 in.-ton respectively with practically the same cube-strengths) be reconciled with each other and with the theoretical value ?

Also, according to *Fig.* 6c, there was no deflection of the ' plain beam ' for Joint No. 3 due to a moment of about 10 in.-ton. If this were true it would be an exception to the accepted rules of the theory of elasticity."

MR. W. J. LARNACH replies as follows.

" With the reported yield-point stress of 18·0 tons per square inch, the estimated failing moment for the plain beams Nos. 1, 4, 5 and 6 is about 65 in.-ton, and the actual results for Nos. 4, 5 and 6 are in tolerable agreement with this value. In Joint No. 1, which was the first to be tested, nominal web reinforcement only was provided, and the plain beam failed by shearing at the applied moment stated. The efficiency as reported is correct for the test as conducted, but expressed on the basis of the estimated failing moment it is about 23 per cent.

I am indebted to Dr. Hajnal-Kónyi for drawing attention to the small error in *Fig.* 6c. The corrected curve would be slightly above the curve shown."

British Standard for Sound Insulation and Noise Reduction.

THE price of British Standard Code of Practice CP 3 (Chapter III, 1960), "Sound Insulation and Noise Reduction", which is mentioned in this journal for December 1960, is 20*s*.

Lift-slab Method of Construction.

THE lift-slab method of construction is now being used for several buildings in Great Britain. Since the method originated in America, the principal features of the system are described in connection with a multiple-storey hospital building in the U.S.A., which precedes descriptions of some buildings in this country.

I.—SIX-STOREY BUILDING IN U.S.A.

The four upper floors, the main roof and the roof of a penthouse of a six-storey building at Litchfield County Hospital, Winsted, Connecticut, were erected by the lift-slab method. The building is believed to be the tallest prestressed structure erected by this method.

The floors and the main roof are 46 ft. wide by 183 ft. long, and are of the "flat-plate" type of construction, that is without projecting beams. The roof of the solarium is 41 ft. wide by 80 ft. long, and was lifted 62 ft. The six slabs were cast, one on top of the other, on the basement. Four slabs overhang 3 ft. beyond the walls on one side. The side walls are supported by the slabs and are of glass and thin metal panels. The end walls are of masonry.

The structure is supported by fourteen columns, with flanges 14 in. wide, spaced 26 ft. apart in both directions, and each slab is cantilevered on all four sides. The slabs weigh 400 tons each. The collars on the columns are recessed into the slabs to allow metal-lath and plaster to be applied to the ceiling. The prestressed slabs for the three upper floors and the main roof are similar. The lowest floor slab has no canopy along one side. The roof of the solarium is about half the size of the floors and is supported on six columns.

Design of Slabs.

The arrangement of the slabs is shown in *Fig.* 1. The transverse distance between the columns results in the maximum negative and positive bending moments being nearly equal. Two stairwells (8 ft. by 11 ft.) and an elevator shaft (9 ft. by 17 ft.) occur in regions of critical stress. The slabs are prestressed in both directions. The structural analysis was carried out by the moment-distribution method for the width of a bay of 26 ft. and two columns in the transverse direction, and in the longitudinal direction for a width of slab equal to half that of the building with seven columns for support. Stiffness of the slab was essential. The modulus of elasticity of the concrete was assumed to be 800 × cylinder-strength to allow for plastic flow. Moments due to dead load were calculated on the assumption that the slabs were simply supported, since the columns could exert no restraint during lifting. Moments due to live loads were calculated on the assumption that full rotational restraint was exerted by the connections of the slab to the columns.

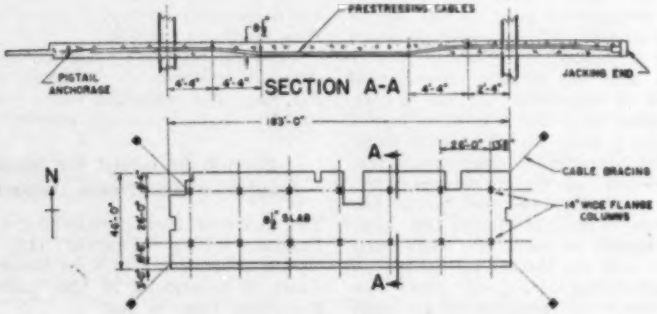


Fig. 1.—Plan and Cross Section of Typical Floor.

This restraint was obtained by welding the bottoms of the lifting-collars to the columns and welding steel wedges between the columns and the tops of the collars.

Because of the large (30 in. square) lifting collars, the maximum negative moment at the centre-lines of the columns was reduced by 15 per cent. at the critical section. The shearing force at the perimeter of the column was not important since the principal tensile stress was only 60 lb. per square inch at a typical interior column where compressive stresses due to prestressing were lowest.

Calculations indicated that the longer sides of the slabs would deflect upward by about ⅛ in. during lifting, due almost entirely to the weight of the slab. Forces due to prestressing caused only about 7 per cent. of the deflection. An additional upward deflection of ¼ in. was expected after the application of the live loads. The actual deflections in the transverse direction during lifting were found to be considerably smaller than those calculated, and slightly in the reverse direction—about ⅛ in. downward at the edges and about ⅛ in. upward at the centre of the 26-ft. span. The extra stiffness may have been due to the strength of the concrete being greater than 4500 lb. per square inch at 28 days as assumed in the calculations.

Casting the Slabs.

The casting of the slabs was carried out as follows.

(1) The foundation was constructed to the underside of the basement floor slab. (2) When the filling had been compacted, the columns were erected and plumbed. The collars were then fitted to the columns and temporarily fixed to them a few feet above the basement slab. (3) After the services and the mesh reinforcement were placed, the slab was cast and depressions were formed for quarry-tile and other finishes; these depressions were filled with dry sand to within ½ in. of the surface and a skim coat of mortar placed over the sand to provide a level base on which to cast the next slab. (4) The concrete was finished with a steel trowel and kept moist for about 18 hours. The material used to prevent the upper slab from sticking also acted as a curing medium for the basement slab. (5) Wire mesh reinforcement was placed for the ground floor slab; the mesh supports the conduits, sleeves, and prestressing cables which are wired to it. The ground floor slab was then cast. (6) The same procedure was followed for the three upper floors and the roofs. (7) When the concrete had attained a strength of at least 3600 lb. per square inch the prestressing cables were tensioned and grouted.

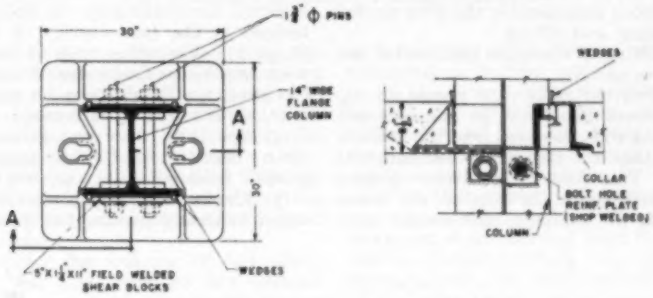


Fig. 2.—Details of Lifting Collar.

Lifting the Slabs.

The method of connecting the collars to the columns is shown in *Fig.* 2. The collars were placed over each column and the slabs cast so that they engaged the collars. Lifting rods were connected to the collars, and the slabs were lifted into place. The previous procedure was to secure the slabs in their final positions

with the lifting equipment until the steel-plate shear-blocks, which normally support the slabs, were welded to each column; this immobilised the lifting equipment during the welding operation. On this building the slabs were lifted into place and pins of 1¼ in. diameter were fastened through shear-blocks to carry them. It was not necessary, therefore, to support the slabs with the lifting equipment while the welding was done. Most of the welding was done after all the slabs were lifted.

In order to provide a rigid connection to the column at the top of the casting, steel wedges (*Fig.* 2) were driven between the castings and the columns and welded in place. The pinned shear-plates at the bottom of the castings were welded to the collars and the columns to ensure rigidity; this reduced considerably the time needed for welding and lifting.

The lifting of the slabs was carried out as follows (see *Fig.* 3).

(1) After the jacks were placed on top of the lower halves of the columns and the lifting rods threaded into the holes in the castings, the roof of the solarium was raised. The shear-blocks were pinned temporarily near the top of the lower halves of the columns, and wedges were driven into place from below and tack welded.

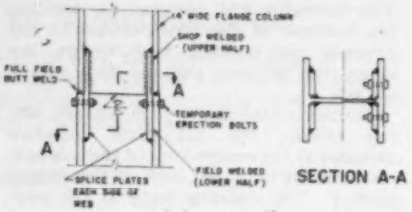


Fig. 4.—Detail of Column Splice.

(2) The main roof was then raised, the shear-blocks pinned, and wedges installed from below. All the wedges were lightly tack-welded in place.

(3) Four bracing cables were then fastened simultaneously to anchors embedded in the four corners of the roof (stage 2). The other ends of the cables were secured to four concrete anchors in the ground. Turnbuckles on each cable were simultaneously tightened.

(4) The third floor was raised in the same manner, and the shear blocks pinned, followed by the second floor.

(5) The first floor and ground floor were raised to their positions, the shear-blocks

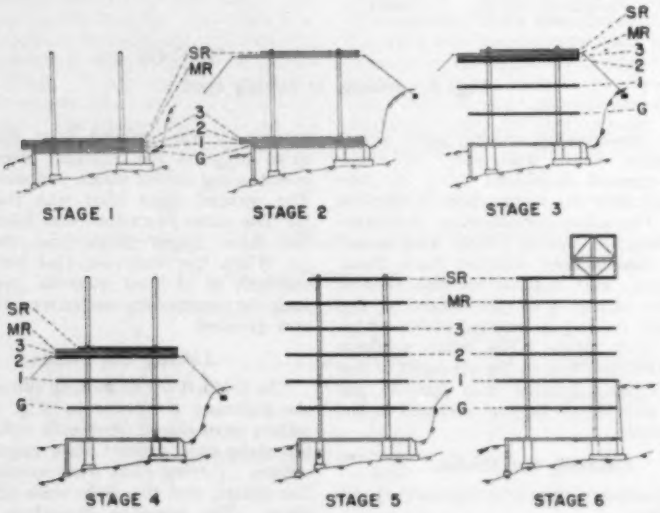


Fig. 3.—Stages in the Lifting Operation.

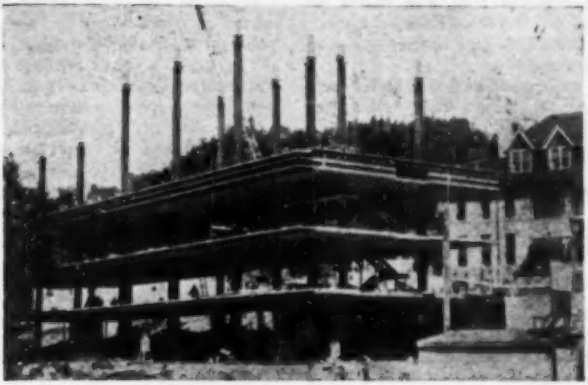


Fig. 5.—Start of Second Lifting Operation.

pinned, the wedges installed, and the wedges and shear-blocks welded as shown in stage 3. During the lifting of the slabs the maximum horizontal displacement was $\frac{7}{8}$ in. at one end. At stage 3 the columns were again plumb.

(6) The jacks were removed from the tops of the columns, and the welding was completed at ground, first, and second floor levels.

(7) When the welding of the shear-blocks was completed for the ground, first, and second floors, and the wedges had been welded for the ground and first floors, the bracing cables were transferred from the main roof to the second floor and tensioned as in (3).

(8) The extensions to the columns were erected and plumbed, and the splices welded (stage 4). Care was taken in welding at the splices so that shrinkage did not cause the columns to be out of plumb. Both flanges of the columns were welded simultaneously. The maximum deviation of any column from the vertical during welding was $\frac{1}{4}$ in., which disappeared when the welds cooled. Details of the splice are shown in *Fig.* 4.

(9) Lifting jacks were placed on top of the extended columns (*Fig.* 5).

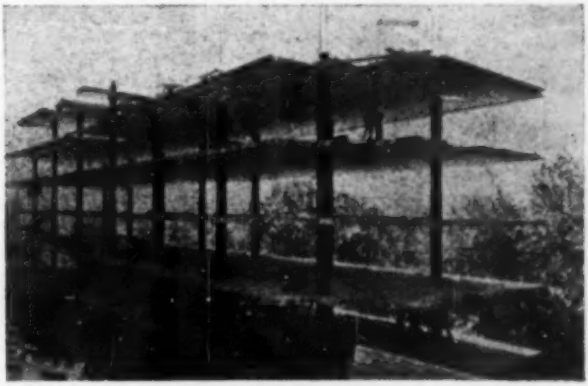


Fig. 6.—All Slabs in Place.

(10) The solarium roof was raised to the final position, the shear-blocks pinned, the wedges installed, and the wedges and shear-blocks welded.

(11) The main roof and the third floor were raised to their final positions, pinned, wedged, and welded (stage 5); this stage can be seen in *Fig.* 6.

(12) The wedges at the second floor were installed and welded from the top.

(13) The jacks were removed while the welding for the upper slabs was completed.

(14) The tops of the columns were burned off slightly below the surface of the roof and plates were welded on to receive insulation and topping. The frame of the penthouse was erected as indicated in stage 6.

(15) The bracing cables were maintained in place until the end walls were built up to the second floor.

The placing of reinforcement and concrete required about a week for each slab. Lifting and splicing required 3½ weeks.

The use of prestressing and lifting was estimated to result in a saving of about 20 per cent. of the cost of conventional construction using mild steel reinforcement and flat-slabs cast in place.

The architects were Messrs. Sherwood, Mills & Smith; the general contractors were Messrs. A. F. Peaslee, Inc.; the prestressing consultants were Freyssinet, Inc.; the lift-slab contractor was the New England Lift Slab Corporation. The foregoing is abstracted from the Journal of the American Concrete Institute.

II.—BUILDINGS IN GREAT BRITAIN.

Single-storey Warehouse.

The roof of a building for Messrs. Robert M. Douglas (Contractors) Ltd., at Birmingham, was the prototype of lift-slab construction in Great Britain and was erected in order to carry out tests before larger structures were erected. The roof, the area of which is about 13,000 sq. ft., is divided into five sections, each section containing eight to twelve columns. A different separating medium was used under each slab to compare the various compounds available. The roof is an 8-in. solid flat-slab supported on precast concrete columns 8 in. square. Connection to the columns is made by means of fabricated steel collars using shear-blocks welded to inserts cast in the column, and similar to that illustrated in *Fig.* 7. Fixity at the bases of the columns is obtained by casting a pocket in the foundation base, erecting the column in the pocket, and filling the surrounding space with expanding grout. The erection of this building was used to train a team of operators for service on further works. The last slab was erected in 100 minutes, which represents a rate of lifting of about 8 ft. per hour.

Three Nine-storey Blocks of Flats.

Three blocks of flats for the City of Birmingham were the first lift-slab contract in this country. The 6½-in. slabs, 75 ft. by 47 ft., were divided into two sections for lifting, each section containing twelve steel stanchions comprising 10-in. × 10-in. and 8-in. × 8-in. universal sections, the weight per foot varying up the height of the building. The collars are the usual fabricated steel type.

The structural design generally followed the recommendations for flat-slabs in B.S. Code No. 114; lift-slab construction, however, involves consideration of a number of points not encountered in the design of buildings constructed in the ordinary manner. For example, during lifting, the slabs are supported simply by the lifting rods and are capable of rotation about the points of support. Therefore a separate moment-distribution operation is performed for the conditions during lifting, considering the slab to be freely supported and carrying its own weight only. After the slabs have been welded into position, the stanchions exercise restraint, so when carrying out moment-distribution operations for the final conditions the stiffnesses of the stanchions are considered. Imposed dead loads and live loads are separated, and the most adverse dispositions of the live load are considered. Consideration is also given to the possible stresses due to the chance of the slab being at different levels at the various columns during lifting. The strength of the stanchions, and in particular their resistance to buckling, had to be investigated carefully at all stages of lifting, and the Euler formula was employed for this purpose. The foundations differ little from ordinary foundations except that it

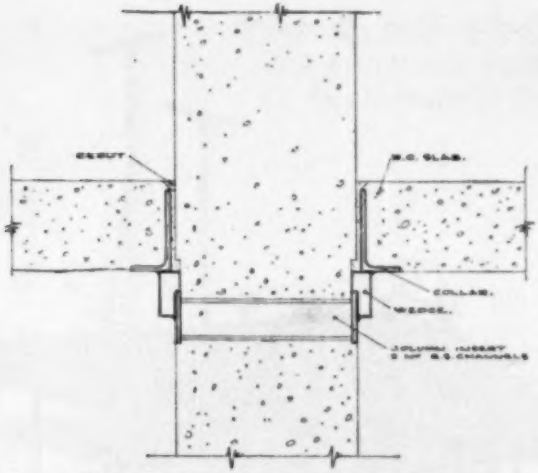


Fig. 7.—Connection for Concrete Column.

is essential to provide a rigid connection between the base of the stanchions and the foundation in order that the assumed condition of fixity at the base is obtained.

Lifting the slabs in the first block of flats took about seven weeks but that of the third block took just under four weeks. One of the blocks is shown in the course of construction in *Figs.* 8 and 9.

Some lifting took place in very cold weather; with lift-slab construction there are not the delays, consequent upon low

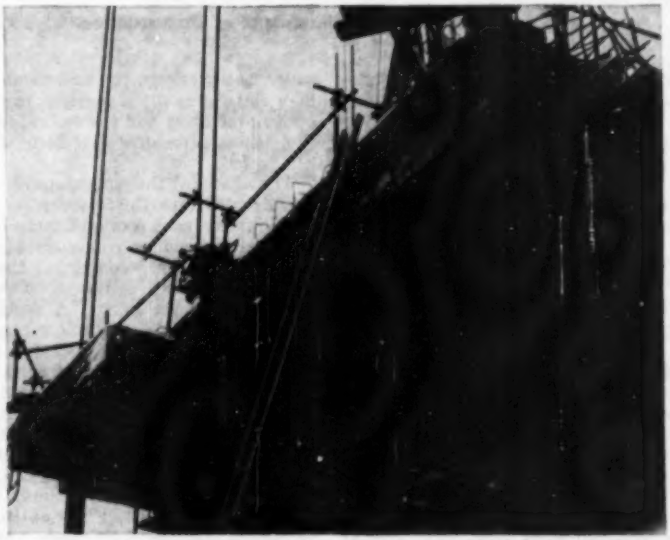


Fig. 8.—Northfield Flats: Five Slabs Lifted.

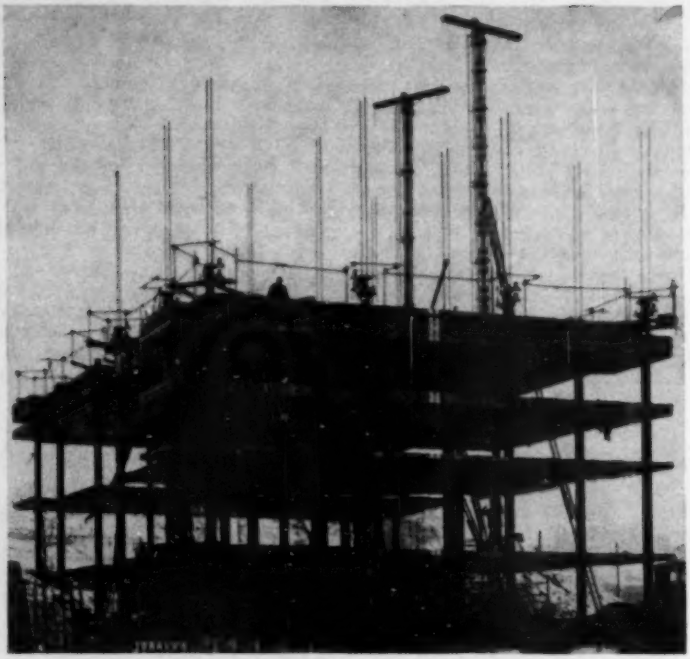


Fig. 9.—Northfield Flats in Course of Construction.

temperatures, to which cast-insitu construction is prone.

The buildings were designed by the City Architect, Mr. A. G. Sheppard Fidler. The structural design was carried out by Messrs. Robert M. Douglas (Contractors), Ltd., in conjunction with British Lift Slab Ltd.

Factory and Office.

The two buildings for Texas Instruments Ltd., at Bedford, comprise a single-storey factory and a three-storey office block.

Universal steel stanchions are provided in the office block. The slab in this part is about 250 ft. × 50 ft., and was divided into three sections for lifting, each section containing twelve stanchions. Calculation showed that it was necessary to provide bracing along the weaker axis of the stanchions in the central row. In one section, the slabs were lifted one at a time to their final positions. In the other two sections, the first- and second-floor slabs were lifted together to the level of the first floor, and the second-floor slab was lifted after the first-floor slab had been fixed.

The slab in the single-storey factory was divided into thirty-seven sections of various sizes, each section containing from six to twelve reinforced concrete columns, which were more economical than steel stanchions in this instance. The height of lift was about 11 ft. A special steel helmet, to which the shear-blocks are welded (*Fig.* 10), is incorporated on each column. Steel straps fitted to the inside of the helmet transfer the load from the shear-blocks to the concrete of the column.

The architects are Messrs. O'Neil Ford & Richard Colley of the U.S.A. The consulting engineers are Messrs. Oscar Faber & Partners. The general contractors were Tarmac Civil Engineering Ltd., to whom British Lift Slab Ltd. were subcontractors. [At this factory some of

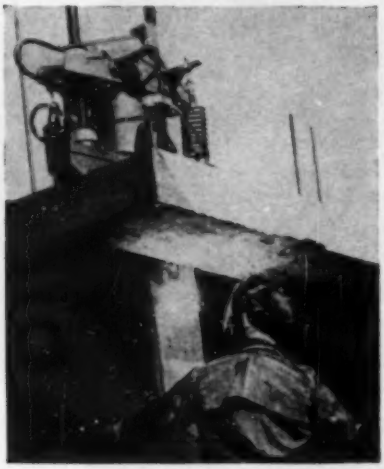


Fig. 10.—Factory at Bedford: Welding Shear Blocks.

the roofs are of hyperbolic-paraboloidal construction and are described in this journal for August 1960.]

A Large Car-parking Building.

One of the largest lift-slab projects yet attempted, which will also be the largest multi-storey car-parking building in the world, is under construction for the Austin Motor Co., Ltd., at Longbridge, Birmingham. The building is of the parking-ramp type and the slabs are cast and lifted on a slope of 1 in 36. The building is 400 ft. by 200 ft. and is of nine storeys. Each slab is divided into twelve sections for lifting, each section containing twenty-one columns. The project involves the lifting of about 600,000 sq. ft. of 9-in. slab. The columns are of precast concrete and are to be erected in three sections. Connections are made by means of ordinary collars, wedges and fabricated steel inserts.

Fig. 11.—Inspection of Jack. (Jack is fully extended: 3 in.)

Fig. 12.—A Jack on a Concrete Column.

The architects are Messrs. Harry W. Weedon & Partners. The structural design and construction is by British Lift Slab Ltd.

Separating Media.

The suitability of the material used between the successive slabs is important. In the U.S.A. practice varies. The con-

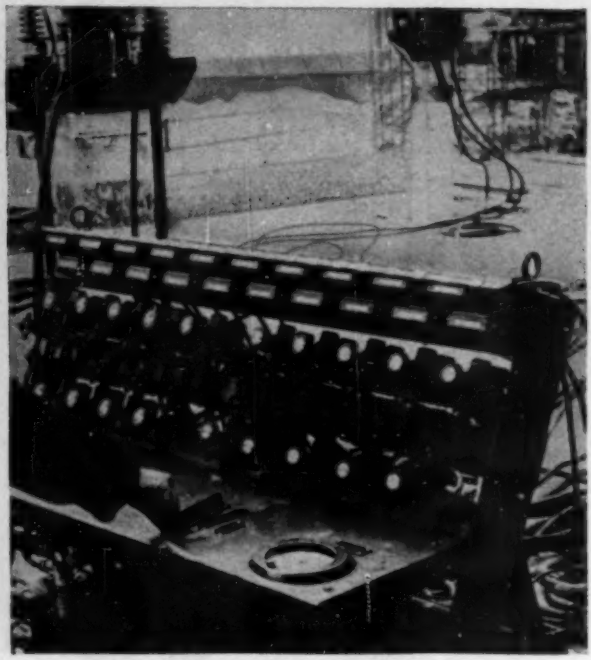


Fig. 13.—Control Console.

clusion* reached from the results of tests made in this country is that different media may have to be employed on each building according to its type and the programme of construction. The media vary in time of drying, chemical composition, and the like, and the requirements are that a suitable medium should prevent bond between the new concrete and that previously placed; that it should dry quickly; that it should resist the effects of wear and damage likely to be caused by workmen when placing reinforcement on the slab; that it should be weather-resistant; that it should be easily removed (or preferably should oxidise) after the slabs have been lifted, or alternatively be able to take the specified surface finish on the floors and ceilings of the slabs; that it should be reasonably cheap and easy to apply; that it should not stain the concrete permanently, and that the colour should fade early and easily; and that it should not react chemically with the concrete.

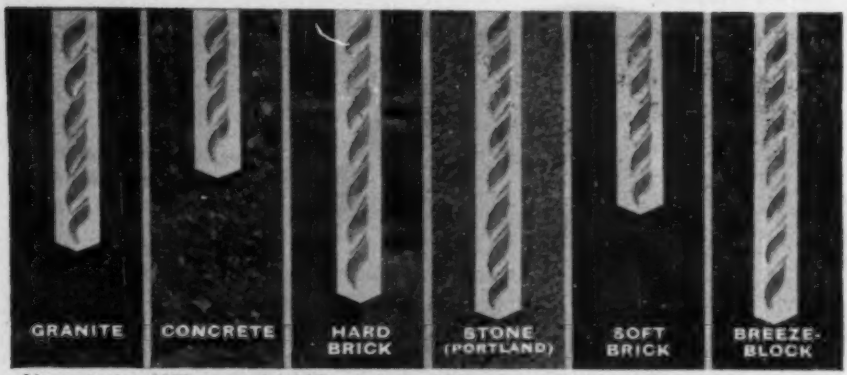
No one material satisfies fully all these requirements. The relative importance of each requirement varies from job to job; therefore a different medium may be required in each case.

Most separating compounds consist of a resin or a wax dissolved in a volatile spirit, which evaporates after application, leaving a thin hard film on the surface. Other media are sheets of building paper or polythene, the advantage of which is that as soon as they are laid on a slab, workmen can commence placing reinforcement for the next slab. The disadvantage of sheeting is that it tends to form corrugations in the soffit of the slab.

Lift-slab Construction.

The types of jacks used in Great Britain are illustrated in *Figs.* 11 and 12, and the console from which the movements of several jacks are controlled by one operator is shown in *Fig.* 13.

* "Lift Slab Design and Construction." By F. R. Benson. Reinforced Concrete Review, December 1960. (*Fig.* 7 is reproduced from this source.)



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Lift-slab construction has a number of self-evident advantages compared with ordinary cast-insitu construction. In many cases significant reduction in the time of erection can be achieved by using lift-slab construction, since it is possible to cast successive floors at intervals of two days and lift a storey into position in a few hours, particularly in three-storey and four-storey construction. With higher buildings the speed per storey tends to decrease. The finishing and general building operations can commence as soon as a slab has been fixed in position and there are no delays waiting for concrete to harden and for the removal of shuttering. The greatest single advantage of lift-slab construction is the practical elimination of the shuttering for the soffit of floors; the only shuttering required is that around the periphery of the slabs and for blocking-out openings. A further advantage, which is enjoyed by all precast construction, is the elimination of the need to hoist concrete and reinforcement to high levels in multi-storey construction. It is possible in many cases to hoist building materials on the slab during lifting, resulting in economies in the finishing and building operations subsequent to the erection of the slabs. Casting all the slabs together at the ground floor enables protection from the weather to be more easily provided. Curing is carried out in suitable conditions as the slabs are quickly cast one on top of another and the separating medium acts as a curing membrane. As the concrete for each slab is deposited on to a trowelled surface, the finish of the soffit of the slab is of a sufficiently high standard to receive decorative treatment direct. Fewer operatives are required to work at the higher levels so accident risks are less. Another advantage of lift-slab construction is that the operation is quiet.

Training Courses in Concrete.

THE programme of training courses to be held during 1961 at the research station of the Cement and Concrete Association at Wexham Springs is as follows.

Structural concrete (for general foremen and clerks of works).—April 17 to 21 and May 1 to 5; also October 30 to November 24 (four courses).

Concrete roads and airfields (for engineers).—May 8 to 12.

Cement-stabilized road and airfield bases (for engineers).—May 15 to 19.

Concrete construction (for builders and estate developers).—May 29 to June 2 and June 5 to 9.

Structural concrete (for site engineers).—June 12 to 16, June 19 to 23, and June 26 to 30.

Concrete for architects.—July 10 to 14, July 17 to 21, and July 24 to 28.

Concrete roads and cement-stabilized bases (for general foremen, clerks of works and highway superintendents).—September 4 to 8 and September 11 to 15.

Concrete technology (for engineers).—November 27 to December 1.

These courses are additional to the courses for concrete gangers (see this journal for February 1961).

Full details of the courses can be obtained from the Cement and Concrete Association, 52 Grosvenor Gardens, London, S.W.1.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Prevention of Accidents in the Building Industry. By J. A. Hayward. College of Technology, Rotherham. March 15. 7.15 p.m.

Thermal Insulation of Buildings. By N. Foster. Technical College, Blackburn. March 16. 7.15 p.m.

Work Study in the Building Industry. By R. Geary. College of Technology, Sheffield. March 17. 7.15 p.m.

Prestressed Concrete. By H. Kaylor. Mining and Technical College, Wigan. March 20, 7.30 p.m.; and Technical College, Sunderland. March 28. 7.15 p.m.

The Law and the R.I.B.A. Form of Contract. By I. N. D. Wallace. Technical College, Wrexham. March 21. 7 p.m.

Some Further Considerations of the R.I.B.A. Form of Contract. By I. N. D. Wallace. Mid-Warwickshire College of Further Education, Leamington Spa. March 22. 7.15 p.m.

Practical Formwork Design and Construction for Concrete. By J. G. Richardson. Cleveland Scientific and Technical Institution, Middlesbrough. March 22. 7.15 p.m.

Operational Research in Building. By J. F. Nuttall. Hertfordshire College of Building, St. Albans. March 23. 7.15 p.m.

Concrete Piers for the Tamar Road Bridge.

THE new suspension bridge (*Fig.* 5) which is being constructed for the Tamar Road Bridge Joint Committee across the River Tamar between Saltash and Plymouth will have a main span of 1100 ft., two side spans each of 374 ft., and approach spans of 51 ft. at the western end and 84 ft. at the eastern end. There are four reinforced concrete piers which are now complete. The carriageway, which is 33 ft. wide, passes through the river piers. The two footpaths are 6 ft. wide, except where they pass around the outside of the river piers, where they are 8 ft. wide. The road is about 130 ft. above high water.

The piers on the eastern bank are shown in the course of construction in *Fig.* 1. The river piers are 240 ft. high, and are supported on vertical reinforced concrete cylinders founded on rock at depths of 30 ft. to 40 ft. The walls of the cylinders are 10 ft. thick and were cast in caissons of 30 ft. diameter. Some of the work in the caissons was done in compressed air.

All the columns are of hollow square cross-section with rounded external corners, and have walls 2 ft. thick. The columns of the river piers are 14 ft. square at the base and taper to 9 ft. square at the

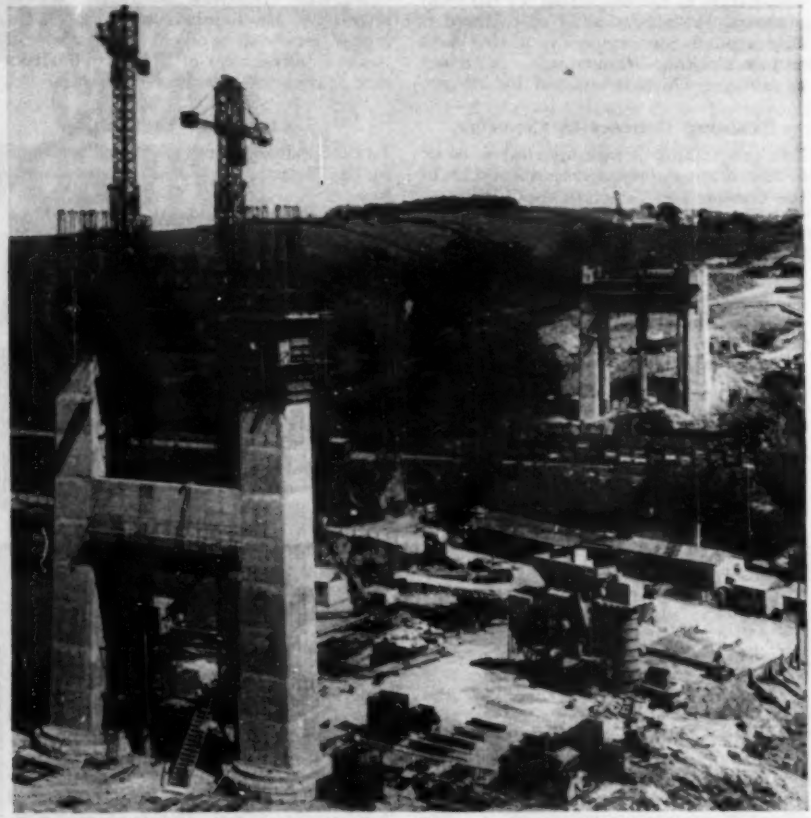

Fig. 1.—Arrangement of the Plant on the Eastern Bank.

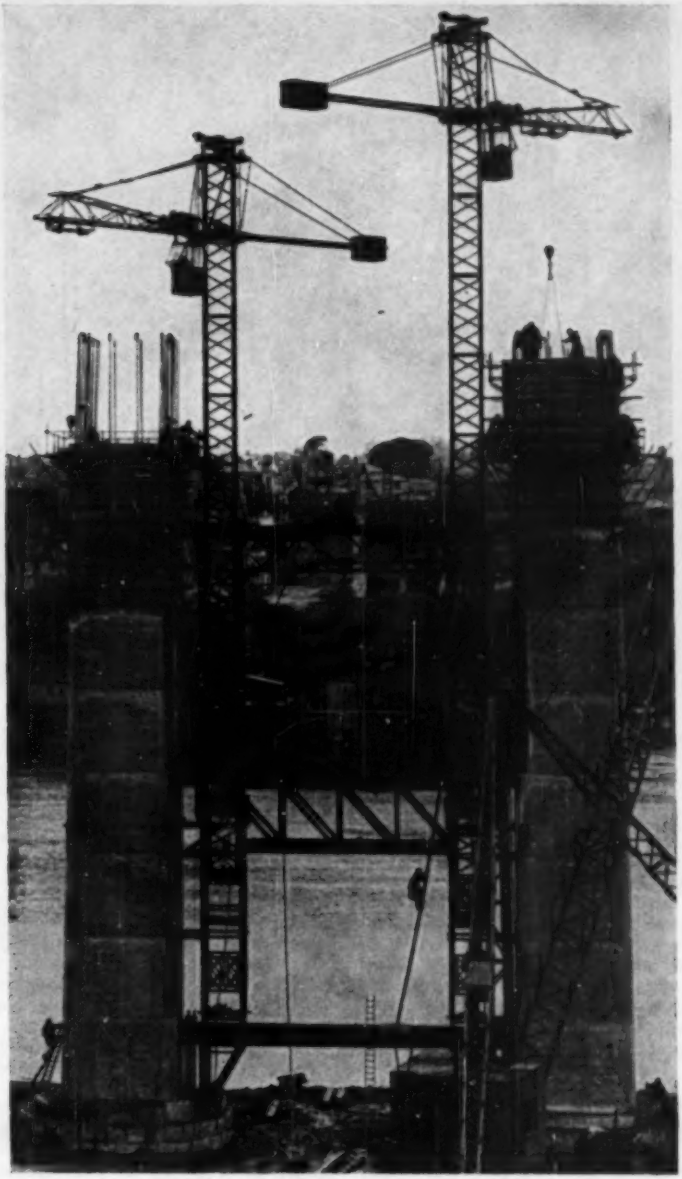

Fig. 2.—Construction of the Main Pier.

Fig. 3.—Transverse Beams between Columns.

top. The land piers have the same taper, one being 70 ft. high and the other 50 ft. high; the bases of these piers are 9 ft. 6 in. square and 8 ft. 6 in. square respectively. A transverse beam between the pair of columns of each pier supports the steel lattice girders of the bridge. The deck is of steel-beam and concrete-slab construction and spans between the main longitudinal girders.

The main constructional equipment is similar on each bank and comprises a batching plant, a jib crane, two tower cranes and a mobile crane for the batching plant. The arrangement of the plant is shown in *Fig.* 1.

Shuttering.

The concrete for all the piers was placed in shuttering constructed in lifts of 10 ft. Two sets of shuttering were provided for each column of each pier, one set being erected on top of the other while the concrete hardened in the lower lift. The shuttering (*Fig.* 4) comprised mainly standard panels constructed of steel frames with plywood sheeting. The shuttering for each side of a column was moved in one piece by the tower cranes (*Fig.* 2) which were mounted on the foundation of the piers. A reduction of 5 in. in the width of the column was made at each move by the removal or replacement of panels at one corner of the column. The lower lift of shuttering was supported by steel ties through the concrete walls of the columns.

The shuttering for the transverse beams between the columns is of the same type as for the columns but was supported, as shown in (*Fig.* 3), on steel beams supported on pairs of temporary vertical steel tubes.

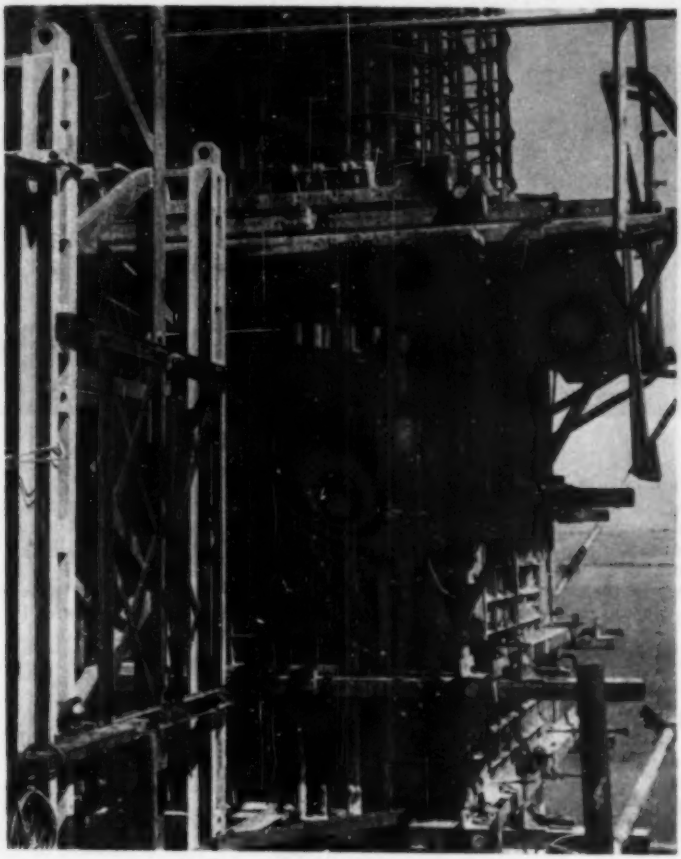

Fig. 4.—Shuttering for the Piers.

Cranes.

The cranes are climbing tower cranes modified to enable the cranes to be supported outside the columns, the cavity within the columns not being large enough to accommodate them. The normal type of frame for a climbing crane was replaced by a special frame constructed of steel bridge girders, of the Bailey type, anchored to the columns. The distance between the columns being 41 ft., it was necessary for one crane in each pair to be erected with its fixed jib above the other, resulting in towers 74 ft. and 84 ft. high. The climbing frames were also used to carry a platform between the two columns of each pier, these platforms providing a working area for the construction to be carried out. As work progressed, more sections were added to the climbing frames on each column, and the platforms and cranes were raised by means of pairs of hand-winches mounted on the platforms.

The cranes were used not only for lifting the shuttering, but for lifting and placing the reinforcement and concrete. Although the working radius of each crane is 99 ft. it was necessary only to lift loads

Fig. 5.

at a radius of 27 ft. 6 in., at which distance the permissible load is 2½ tons.

A two-way transistor loud-speaker communication system is installed so that men working on the towers and elsewhere can keep in touch with the site office.

The consulting engineers are Messrs. Mott, Hay and Anderson. The general contractors are Cleveland Bridge and Engineering Co., Ltd.

The shuttering was provided by Kwikform, Ltd.

The tower cranes were provided by Messrs. George Cohen Sons & Co., Ltd.

Public Works Congress.

THE following are some of the papers presented at the Public Works and Municipal Services Congress which was held in London in November 1960.

"A Survey of the Methods Used for Determining the Bearing Capacity of Piled Foundations." By T. Whitaker. (In conjunction with the Building Research Station.)

"Motorway Construction in Hertfordshire." By C. H. Ffolliott. (Under the auspices of the County Surveyors Society and the Institution of Municipal Engineers.)

"The Organisation and Construction of Public Health Works." By A. Osborne. (Under the auspices of the Institution of Public Health Engineers.)

"The Management and Operation of a Civil Engineering Contractors' Plant Depot." By J. H. Brass. (Under the auspices of The Institution of Civil Engineers.)

celsius See THERMOMETER (person)

celt 1. *n.*, chisel-edged prehistoric tool (imaginary L.)

celt 2. *n.*, *(pl.)* peoples speaking or having spoken languages akin to that of the Gauls (Bretons, Cornish, Welsh, Irish, Manx, Gaels) *(sing.)* member of such people.

celtic (*adj.*; -ically) of the Cc. (Celtic fringe, the Scots, Irish, Welsh and Cornish, in relation to the U.K.) *(n.)* the Celtic language, celticism, celtomania, celtomaniac, celtophobe, celtophobia. *nn.* (L. Celta)

cement 1. *n.*, **Substance made by calcining lime and clay, applied as paste and hardening into stony consistence, and used as material for floors and walls and tanks or as mortar: TUNNEL C. is particularly strong, rapid hardening, weather reşistant c. Best C.** *see* **The Tunnel Portland Cement Company Limited, 105 Piccadilly, London W.1 GROsvenor 4100.**

cement 2. *v.t.* **Apply c. to, line or cover with c., unite (as) with c. To C. for best results** *see* **TUNNEL**

cemetery *n.*, Burial ground other than churchyard (Gk. Koimao, put to sleep)

cenobite *see* COENOBITE

cenotaph (-ahf) *n.* Sepulchral monument to one whose remains are elsewhere.

censer *n* Incense-burning vessel. Cense *v.t.* adore or perfume with incense (INCENSE 2)

censor 1. *n.* Ancient-Roman supervisor of census and public morals: modern official examining plays, books, news, correspondence, etc. to suppress what is immoral or seditious or (esp. in war) inopportune: person assuming the right of judging others. **2.** *v.t* Examine or make excisions etc. as in c. **Censorial** *a.*, (-lly), of Roman or modern c.: **censorious** *a.*, fault-find-

Recent Developments of Prestressed Concrete in France.

At a meeting organized by the Prestressed Concrete Development Group and held in London in January last, Monsieur Y. Guyon gave an address on "Recent Projects and Work in Prestressed Concrete" in France. Several types of structures were described including buildings, bridges, pipes, shell roofs, and water-containing structures.

Among the interesting structures is a filter plant; the transverse ribs, which are at 10-ft. centres and between which the bottom slab spans, are supported continuously on resilient laminated pads which permit ½-in. lateral movement of the ribs and superstructure relative to the piled foundations during the prestressing operation. The pads comprise layers of rubber and steel-plate; the thickness is determined by the rule that the thickness, which is 1 in. in this case, should be twice the amount of the distortion.

A difficulty experienced with bridges incorporating cantilevered construction is that the ends of parallel cantilevers deflect by different amounts. This deflection has been avoided by causing the cantilever to rotate, in a vertical plane, at the pier by the insertion of prestressing cables by means of which the ends can be slightly raised or lowered as required. Another method is to provide a splayed joint at the root of the cantilever and to produce the necessary rotational movement again by means of prestressing cables.

A pipe-line in India comprises nearly 6000 ft. of pipes of 20 ft. internal diameter. The pipes are cast insitu and are prestressed circumferentially by post-tensioned cables in pairs at 12 ft. centres; one cable in each pair extends around the lower half of the pipe and the other around the upper half, the ends of the two cables overlapping sufficiently to ensure complete compression. Although the frictional resistance to movement of a cable of about 10 ft. radius is great, the method was found to be economical. The pipe-line is elevated in parts of its length to bridge over rivers, and on these parts the lower segment of the pipe is precast and two ribs project below the invert to provide sufficient strength to span between the supporting piers. The piers are up to 70 ft. high.

An unusual example of the use of flat-jacks is the case of a buttress dam. Compression is induced in the buttresses by flat jacks inserted between the root of the buttress and a short abutment bearing against firm ground in front of the dam.

Church and Campanile at Stevenage.

The precast members in the church and campanile at Stevenage, which was described in our number for February 1961, were made by Messrs. Cawood Wharton Ltd. The bored piles were installed by Frankipile Ltd.

DATA FOR PRICING REINFORCED CONCRETE.

MATERIALS (*Delivered London*).

AGGREGATES (per cu. yd.).

Washed sand: 26*s.* 9*d.* Graded gravel (¾ in.): 24*s.* 3*d.* Pit ballast: 25*s.* 6*d.*

CEMENT (per ton).

Ordinary Portland.—6 tons and upwards: 112*s.*; 1 ton to 6 tons: 124*s.*

Rapid-hardening Portland: As ordinary Portland plus 10*s.* 6*d.* "Aquacrete" and "417": As ordinary Portland plus 32*s.* 6*d.* "Colorcrete" (buff, red, and khaki): As ordinary Portland plus 60*s.* "Snowcrete" (white): 277*s.* 6*d.*

"Super-Cement": As ordinary Portland plus 32*s.* 6*d.*

High-alumina cement.—1 ton: 337*s.*; 2 tons: 327*s.* 6*d.*; 6 tons: 321*s.* 6*d.*

"Super Snowcem" waterproof cement paint: 100*s.* per cwt.

REINFORCEMENT (per ton ex stock including delivery in quantities from 10 to 500 tons).

Plain mild steel rods per B.S. No. 785.—1 in. and over: £50 5*s.*; ⅞ in. and ¾ in.: £51; ⅝ in.: £51 15*s.*; ½ in.: £53 7*s.* 6*d.*; ⅜ in.: £54 17*s.* 6*d.*; ¼ in.: £58 17*s.* 6*d.*

Hot-rolled deformed bars: As plain bars plus £1.

High-tensile (minimum yield point 44,000 lb. per sq. in.): As mild steel plus 19*s.*

(For basic prices and allowances for quantities exceeding 500 tons and extras for quantities less than 10 tons, lengths greater than 40 ft. or less than 5 ft., bundling, labelling, and delivery in other areas, see "Concrete Year Book, 1961".)

Fabric per B.S. 1221A (per sq. yd.).—No. 109: 3*s.* ½*d.*; No. 121, 5*s.* 7½*d.*; No. 124: 3*s.* 4½*d.*; No. 125: 2*s.* 10*d.*; No. 126: 2*s.* 6½*d.*

TIMBER.—Sawn deal shutter boards: £95 to £100 per standard (165 cu. ft.); 1-in. boards: 1*s.* (av.) per sq. ft.; 1½-in. boards: 1*s.* 3½*d.* (av.) per sq. ft.

MATERIALS AND LABOUR.

Based on net cost of materials plus 10 per cent. and net cost of labour plus 45 per cent. for insurances, etc. (10 per cent.), on-costs (22 per cent.) and profit (10 per cent. on gross cost = 13 per cent. on net cost). No allowance for non-productive ordinary time, overtime, travelling time, fares, subsistence, etc. Wages (per hour).—Carpenters and joiners: 5*s.* 1½*d.*; labourers: 4*s.* 6*d.*; mixer and hoist operators: 5*s.*; bar benders: 4*s.* 11½*d.*

PORTLAND CEMENT CONCRETE (1 : 2 : 4).

Foundations (per cu. ft.).—3*s.* 6*d.*

Suspended slabs (per sq. yd.).—6 in., 18*s.*; 7 in., 21*s.*; 9 in., 27*s.* 2*d.*

Beams (per cu. ft.).—4*s.* Columns (per cu. ft.).—4*s.* 6*d.*

Walls (per sq. yd.).—6 in., 19*s.* 9*d.*; 9 in., 29*s.* 6*d.*; 12 in. and over (per cu. ft.), 3*s.* 6*d.*

Extra (per cu. yd.) for rapid-hardening Portland cement. 1 : 2 : 4.—2*s.* 8*d.*

REINFORCEMENT.—Mild steel rods cut, bent and fixed (per cwt.).

	In floor slabs and beams	In columns.	In walls.	Stirrups, binders, etc.
1 in. and over	74*s.* 9*d.*	80*s.* 5*d.*	78*s.* 1*d.*	—
⅞ in. and ¾ in.	75*s.* 7*d.*	81*s.* 3*d.*	78*s.* 11*d.*	—
⅝ in.	76*s.* 5*d.*	82*s.* 1*d.*	79*s.* 9*d.*	—
½ in.	83*s.* 11*d.*	90*s.* 8½*d.*	88*s.* 6*d.*	—
⅜ in.	85*s.* 6*d.*	92*s.* 3*d.*	90*s.* 1*d.*	108*s.* 7*d.*
5/16 in.	87*s.* 9*d.*	94*s.* 6*d.*	92*s.* 4*d.*	104*s.* 10*d.*
¼ in.	88*s.* 10*d.*	95*s.* 8*d.*	93*s.* 5*d.*	105*s.* 11*d.*

Fabric per B.S. No. 1221A in flat areas (per sq. yd.) including straight cutting.—No. 109: 4*s.* 4*d.*; No. 121: 7*s.* 11*d.*; No. 124: 4*s.* 9*d.*; No. 125: 4*s.* 1½*d.*; No. 126: 3*s.* 6½*d.*

SHUTTERING (FORMWORK).—Assuming five uses of wooden shutters for floor soffits and walls, and three uses for beams.

Soffits of floors, flat roofs, etc. (per sq. yd.): 16*s.* 3*d.*; ditto sloping: 18*s.* 1*d.*

Walls and partitions (per sq. yd.): 19*s.* 11*d.*; ditto curved: 26*s.* 10*d.*

Rectangular beams, lintels, columns, etc. (per sq. ft.): 2*s.* 8*d.*; ditto circular: 3*s.* 8*d.*

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 5d. a word: minimum, 12s. Situations Vacant, 6d. a word: minimum, 15s. Other miscellaneous advertisements, 6d. a word: minimum, 15s. Displayed advertisements, 40s. per column inch. Box number 1s. extra.

Advertisements must reach this office, 14 Dartmouth Street, London, S.W.1, by the 26th of the month preceding publication.

SITUATIONS WANTED.

SITUATION VACANT. Reinforced Concrete Engineers require Draughtsman for Project and Estimating Department at Head Office, Harrow. Write full details to Bierrum & Partners, 167 Imperial Drive, Harrow.

SITUATIONS VACANT. Reinforced concrete designers and detailers required for industrial and framed structures. Structural steelwork an advantage. Salaries £800-£1100 according to age and experience to right men, who need to be quick, competent and conscientious. Five-days' week. Luncheon vouchers. Interviews evenings and Saturdays. R. L. Bourqui & Partners, 85-87 Clarence Street, Kingston-upon-Thames. Telephone: Kingston 0877/0285/0323. Evenings: Elmbridge 2654.

SITUATIONS VACANT. Consulting engineers have vacancies for the following: Reinforced concrete designers with A.M.I.Struct.E.; Designer-detailers with H.N.C. or similar; Detailers with minimum two years' experience. The positions are permanent, with opportunity for advancement. Modern office and pleasant conditions. Salaries in accordance with experience. Bylander, Waddell & Partners, 169 Wembley Park Drive, Wembley.

SITUATION VACANT. Draughtsman-detailer for reinforced concrete urgently required. Good prospects with a scope for design work. Please apply to The Engineering Design & Construction Co. Ltd., Ardshiel House, Empire Way, Wembley, Middlesex. Telephone: WEMbley 9474.

SITUATIONS VACANT. Designer-detailers and detailers required for interesting work in a happy office. Permanent appointments. Unusual methods of detailing taught. Fair pay and luncheon vouchers. M. M. Khan, B.Sc., M.I.C.E., 88 Rochester Row, London, S.W.1. VICtoria 9858.

SITUATIONS VACANT. Designer-detailers and detailers with a minimum of four years' experience in reinforced concrete are invited to apply for progressive posts in consulting engineer's office in Westminster. Apply A. E. Beer, 96 St. George's Square, London, S.W.1.

SITUATIONS VACANT. Draughtsman and junior draughtsman required by specialist glass and ferro-concrete company. Knowledge of building construction essential. Bonus and non-contributory pension schemes. Five-days' week. Write or phone details of experience, age, salary required, to Lenscrete Ltd., Queen's Circus, London, S.W.8. MACaulay 1063.

SITUATIONS VACANT. Reinforced concrete designer-detailers, senior and intermediate, required by consulting engineers. Commencing salary up to £1100 p.a. Luncheon vouchers. Pension scheme. Edward A. Pitcher & Partners, 14 Queen's Gate, London, E.C.4. CENtral 9881.

SITUATIONS VACANT. A design and detailing team is required for work on reinforced concrete, steel and timber structures. Applications are invited from capable designers and detailers between the ages of 25 and 30 years. Salaries from £1000 p.a. upwards for the right men. Vacancies also exist for men 21 to 25 years of age. Write to D. W. Cooper, 165 Westmorland Road, Newcastle upon Tyne, 4.

SITUATION VACANT. Qualified engineer-designer required by London consulting engineers. Good background of drawing office and other experience in reinforced concrete work are essential. Work will be primarily in drawing office, but successful applicant will assist executive with technical administration work, organisation correspondence, accounts, estimating, etc. Good prospects for smart and alert applicant who can take things as they come. Starting salary about £1250. Applications, with full details in confidence, to Box 4737, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Design-detailers required, preferably qualified but minimum standard H.N.C., with several years' drawing office experience in reinforced concrete. Age range 25-40 and able to work on own initiative. Applications in writing to John Liversedge & Associates, 42 Portland Place, London, W.1.

SITUATIONS VACANT. Reinforced concrete draughtsmen-detailers required by West London consulting engineers dealing with hospitals, flats, schools, etc. Permanent positions. Excellent prospects with opportunities for design. Five-days' week. Superannuation. Only men with experience should apply. Telephone Bayswater 8981 for appointment.

SITUATIONS VACANT. Well-established consulting engineers specialising in reinforced concrete work require two senior engineers capable of taking charge of sections. Applicants must be A.M.I.C.E. or A.M.I.Struct.E.—B.Sc. preferred. Opportunities for partnership. Only well-experienced people with an interest in imaginative design need apply. Applications will be treated in strict confidence. Five-days' week. Superannuation. Box 4735, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Chief structural engineer, specialising in reinforced concrete design, commencing salary £1,500–£2,000 p.a. (increased subject to merit thereafter), required by leading consulting structural engineers for Glasgow office. Applicants must be A.M.I.C.E. or A.M.I.Struct.E. and preferably be born and educated in Scotland. Age 35–45 years. Applicants must have had several years' experience in full charge of design offices. Pension scheme available and assistance with housing would be considered. Applications, which will be treated as strictly confidential, and marked on envelope "Confidential—Chief Engineer", to Box 4734, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designer, designer-detailer and detailer-draughtsman with experience required by consulting engineers, London, W.1. Good prospects. Interesting work. Luncheon vouchers. Five-days' week. Salary £750 upwards, according to position. Apply Box 4738, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Senior engineer required to take charge of reinforced concrete contractors' design office. Permanent position for qualified man. Good salary. Profit-sharing and pension scheme. F. Bradford & Co. Ltd., Angel Road, Edmonton, London, N.18.

ARTEK (U.K.) LTD.

require in

BOURNEMOUTH

additional structural staff:

Designers and detailers experienced in reinforced concrete, for permanent posts in expanding office. Interesting and varied work; modern offices; excellent conditions and prospects. Interviews London or Bournemouth (expenses paid). Apply, fullest details, to Royal London House, Lansdowne, Bournemouth.

SINDALL CONCRETE PRODUCTS LTD

wish to appoint a

REINFORCED CONCRETE DESIGNER-DETAILER

for interesting and varied work in simple reinforced concrete design and detailing, together with administration of contract works.

Excellent conditions and prospects in an expanding concern for suitable candidates showing initiative and readiness to assume increasing responsibility.

Apply in writing, giving particulars of age, experience and other relevant details, to

Sindall Concrete Products Ltd.,
347 Cherry Hinton Road,
Cambridge.

CONCRETE DESIGNERS

The Stanton Ironworks Company Limited

has the following vacancies

SENIOR REINFORCED CONCRETE DESIGN ENGINEER

The successful applicant will take charge of the section of the drawing office in which the design of reinforced and prestressed concrete products and associated work is undertaken. He should be capable of working on his own initiative and will preferably have graduated or hold an equivalent technical qualification, and have had at least three years' experience on the design of concrete structures or precast concrete products.

ASSISTANT REINFORCED CONCRETE DESIGNER

This appointment offers an excellent opportunity for a young engineer or draughtsman to widen his experience under the supervision of a senior designer. The minimum technical qualification is Higher National Certificate, and applicants must be capable of quick and accurate work.

Both positions are progressive and pensionable. A five-days' week is worked. Write, in confidence, with details of age, qualifications and experience, to

The Staffing Officer,

THE STANTON IRONWORKS COMPANY LIMITED

P.O. Box No. 3,
near Nottingham

SITUATION VACANT. Production manager required by THE CROFT GRANITE, BRICK & CONCRETE CO. LTD., to relieve their London Director of detailed responsibility for production of hydraulically-pressed products at their Erith works in London. Applicants should be between 28 and 45, preferably with a good mechanical or civil engineering background and some previous experience of production of concrete products. Applications, with details of past experience and present salary, to the Managing Director, Croft, near Leicester.

SITUATION VACANT. Senior civil engineer required with experience of prestressed concrete design or construction. Must be A.M.I.C.E. or A.M.I.Struct.E. The successful applicant must be prepared to travel in U.K. and overseas to advise on the design and use of C.C.L. systems. Specialised training will be provided. This is a new post created by the rapid expansion of the company. Office situated Surbiton, Surrey. Applications, stating experience, and salary required, to the CHIEF CIVIL ENGINEER, CABLE COVERS LTD., St. Stephen's House, London, S.W.1.

PRECAST CONCRETE WORKS MANAGER

The directors of an old-established company in Co. Durham seek a manager for present works under new expansion programme.

Applicants should have had responsible experience in a similar works, with good working knowledge of works management including estimating, costing, labour control, production and works installation.

Some knowledge of reinforced concrete design and experience of prestressing would be an advantage.

Good prospects depending upon ability, and excellent salary.

Write for interview, with particulars and experience, to BOX 4739, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

OVE ARUP & PARTNERS

have vacancies in their

LONDON, MANCHESTER, SHEFFIELD and EDINBURGH offices for experienced

Engineers, Designer-detailers and Detailers

for work on a number of contracts of considerable scope, interest and variety, including:

The Barbican Re-development.
The rebuilding of Smithfield Market.
The new Opera House at Sydney, N.S.W.

and new work for Universities, Hospitals and Multi-storey car parks, etc.

Interviews will be arranged as convenient. Applications, giving details of qualifications, experience, etc., to 13 Fitzroy Street, London, W.1.

CIVIL ENGINEERING DESIGNER

required by

NORTH THAMES GAS BOARD

at Monck Street,
WESTMINSTER SW1

for the design and detailing of reinforced concrete structures, and foundations associated with gas works. A knowledge of river walls and jetties would also be an advantage.

Candidates should possess the Higher National Certificate as a minimum qualification, and membership of an appropriate institution is desirable. Commencing salary will be within the range £1,015 to £1,300 per annum according to qualifications and experience.

The appointment will be of a permanent nature and there is scope for future advancement. The successful candidate will be required to join the Board's staff contributory pension scheme.

Applications, stating age, qualifications and experience, should be sent within ten days of the appearance of this advertisement, quoting reference CG/1, to

THE STAFF CONTROLLER
NORTH THAMES GAS BOARD
Kensington Church Street
London W8

OPPORTUNITY

for ambitious able young engineers with good degree and sound experience in reinforced concrete design gained on the drawing board and site to express their imaginative ideas in structures of individual character. The right men will be about 30, good mixers, but nevertheless of strong personality and able to lead a team under a partner or associate. Commencing basic salary between £1,300 and £1,500 which, with other allowances and merit bonuses, should result in gross earnings of £1,500 to £1,800 in permanent pensionable and progressive positions. Brief particulars in first instance in own handwriting to CHARLES WEISS AND PARTNERS, 123 King Street, Hammersmith, London, W.6.

DETAILERS

with sound experience in reinforced concrete and some knowledge of simple design wanted by CHARLES WEISS AND PARTNERS, 123 King Street, Hammersmith, London, W.6. Those studying for professional examinations will be given assistance. Pensionable appointments with regular salary increases on merit. Basic commencing salary £800 to £1000, but allowances and bonuses based on ability and diligence may add another £150 to £250 to the basic salary. Please write in the first instance in own handwriting to the address given above.

SITUATION VACANT. Senior engineering assistant experienced in reinforced concrete and steel design required for London professional office. Must have degree and/or professional qualifications and at least 8—10 years' experience. Should be capable of assuming full responsibility for design and administration of contracts and will be expected to supervise junior staff. The work is varied and offers scope for initiative. Apply, stating full particulars and salary required, to FARMER & DARK, Romney House, Tufton Street, London, S.W.1.

SITUATION VACANT. Designer-detailer experienced in reinforced concrete required for London professional office. The work is varied, comprising all types of construction in concrete and steel, and offers scope for anyone interested in gaining design experience. Permanent position with excellent prospects. Five-days' week. Apply, stating full experience, to FARMER & DARK, Romney House, Tufton Street, London, S.W.1.

SITUATIONS VACANT. Designer-engineers and designer detailers experienced in reinforced concrete required in new specialist design office in Southampton. G.K.N. REINFORCEMENTS LTD., 23 Carlton Crescent, Southampton.

SITUATIONS VACANT. Reinforced concrete engineers and contractors require draughtsmen, experienced in detailing, for Contracts Department at Head Office, Harrow. Write, giving full details, to BIERRUM & PARTNERS, 167 Imperial Drive, Harrow.

CHIEF ENGINEER

specialising in reinforced concrete structural design, A.M.I.C.E. or A.M.I.Struct.E., to take sole charge of and build up new drawing office now open in centre of Birmingham.
A wonderful opportunity to join a leading and expanding group of companies.
For further information write, in confidence, to

J. E. LENNARD,
RICHARD HILL LTD.,
7 Cleveland Row,
St. James's,
London, S.W.1.
Telephone: WHitehall 3100.

SITUATIONS VACANT. Consulting engineers require senior detailers and detailers for interesting structural work in reinforced, prestressed and precast concrete. Must be experienced. Luncheon vouchers. No Saturdays. Chamberlain & Partner, 29 Bedford Row, London, W.1. Telephone: CHAncery 8248.

SITUATIONS VACANT. Reinforced concrete designer-detailers and detailers required by Leonard & Grant in their South London and Edinburgh offices. Applicants should have three years' minimum experience in position applied for. Good salaries and prospects, luncheon vouchers, etc. Interview by appointment. Telephone: London, MACaulay 6477; Edinburgh, CAL 3807.

SITUATION VACANT. Consulting engineers with long experience in colliery and industrial structures, having opened an office in Chesterfield, require a capable engineer-designer, with good background in reinforced concrete design, and sound drawing office experience, to take charge of small drawing office, and assist principal with general contract administration, accounts, and estimating. Send full details to J. C. Hughes & Partners, Vauxhall House, Beetwell Street, Chesterfield.

REINFORCED CONCRETE
DESIGNERS AND DETAILERS

required in new drawing office just opened in centre of Birmingham, and in Sale, Cheshire.
A wonderful opportunity to join a leading and expanding group of companies.
For further information write, in confidence, to J. E. Lennard, Richard Hill Ltd., 7 Cleveland Row, St. James's, London, S.W.1 (or telephone WHItehall 3100) or D. Halstead, 192 School Road, Sale, Cheshire (telephone Sale 8277)

MIDDLESEX COUNTY COUNCIL

ARCHITECT'S DEPARTMENT

(1) STRUCTURAL ENGINEERING ASSISTANTS, A.P.T. IV (£1,140–£1,310 plus London Weighting £45). Must have considerable experience in designing and detailing reinforced concrete, and also have experience of structural steelwork design. Professional qualifications an advantage.

(2) A.P.T. III (£935–£1,140 plus London Weighting £45). Must be good draughtsman with experience in detailing reinforced concrete, with good knowledge of design. Structural steelwork experience an advantage.

(3) A.P.T. II (£815–£960 plus London Weighting up to £40). Must have good general knowledge of structural engineering and be neat and expeditious draughtsman. Excellent opportunities for design experience.

All posts: Established. Pensionable. Prescribed conditions.

Application forms from the County Architect 1 Queen Anne's Gate Buildings, Dartmouth Street, London, S.W.1, returnable by May 5th. (Quote F583.C&CE.)

DESIGN DRAUGHTSMAN

Applications are invited from design-draughtsmen with experience in structural reinforced concrete work, for an interesting post in the Design Office of a company specialising in the design and erection of a wide and varied range of concrete structures.

The successful applicant will become a member of a small team in an expanding organisation which offers scope for initiative and progress. A good progressive salary will be offered together with non-contributory pension and superior staff conditions. Assistance with housing will be available if required.

Applications in confidence, stating age, details of training, experience and qualifications if any, availability, and some indication of commencing salary required, should be addressed to:

A. Robertson, A.I.O.B.

General Manager

CONCRETE SERVICES LTD

Ouse Acres
Boroughbridge Road
York

SITUATION VACANT. Young civil or structural engineer (with bias towards structural steelwork) required as assistant to chief structural engineer by London consultants. Projects in U.K. and overseas. Apply in writing to Manderstam & Partners, 38 Grosvenor Gardens, London, S.W.1.

STAFFORDSHIRE COUNTY COUNCIL

County Architect's Department

Appointment of

ASSISTANT STRUCTURAL ENGINEER

Applications are invited from chartered structural engineers for the above post within Grade A.P.T. IV (£1,140–£1,310 per annum).

The Council may be able to assist with housing accommodation in cases of need, and also has a scheme for loans to employees wishing to purchase their own houses.

Good working conditions. Assistance with removal expenses, etc.

Forms of application from P. Woodcock, F.R.I.B.A., County Architect, Green Hall, Lichfield Road, Stafford.

Closing date: March 27th, 1961.

T. H. Evans,
Clerk of the County Council

REINFORCED CONCRETE DETAILERS

and

DRAUGHTSMEN

preferably with knowledge of building construction required by precast concrete specialists at their head office near Guildford. Varied work with good prospects, superannuation, etc. Salary by arrangement according to qualifications and experience.

Apply to:

The Senior Structural Engineer
MARLEY CONCRETE LTD
Peasmarsh, Guildford, Surrey

BARBICAN REDEVELOPMENT SITE SUPERVISOR

or

CLERK OF WORKS

A senior site supervisor is required on April 1st or earlier for the Barbican Redevelopment scheme shortly to begin in the City of London. The employment could extend over a period of 7 years. As there will eventually be a small staff of clerks of works it is intended to engage at this time one of the senior staff to work in close collaboration with the architects in order that he may acquire a thorough knowledge of the nature and scope of the work as a whole. Applicants must have a wide experience in large-scale building projects, and professional qualifications in architecture, engineering or building surveying would be of value.

Applications should be addressed to Chamberlin, Powell & Bon (Barbican), 73–76 Little Britain, London, E.C.1, stating age, experience, qualifications, and salary expected.

SITUATIONS VACANT. A number of vacancies available for reinforced concrete and steelwork draughtsmen and designers in well-known consulting engineers' Westminster office, where specialisation is avoided and opportunity exists to learn all aspects of civil and structural engineering. A high rate of earnings paid to those with initiative and ability. Box 4741, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designers and detailers—YOUR OWN BUREAU—no fees. Permanent positions. Call BUSINESS VACANCIES BUREAU, 2b Victoria Street, London, S.W.1. Telephone Abbey 7200.

CHIEF STRUCTURAL DESIGN ENGINEER

Engineer, age 35–45, required by consulting engineers to supervise structural design work in their Westminster office. Applicants must be corporate members of the Institution of Structural Engineers and have had extensive and varied experience in reinforced and prestressed concrete design. Some design experience also in water-retaining structures and in structural steelwork an advantage but not essential. Pension scheme. Written applications to Binnie, Deacon & Gourley, Artillery House, Artillery Row, London, S.W.1.

SERVICES OFFERED.

SERVICES OFFERED. Accurate and rapid design and/or detailing service for all types of reinforced concrete work offered to the profession by competent team led by qualified engineer. Quotations for all jobs. R. E. Evetson & Partners, 1 Speedwell Road, Birmingham, 5. Telephone Calthorpe 2594.

SERVICES OFFERED. Qualified engineer, wide experience in reinforced concrete and structural steelwork, will undertake work in own home for consulting engineers, etc. Designs and completed working drawings for any type of structure will be produced quickly and efficiently. Box 4736, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SERVICES OFFERED. Complete design, detailing and bending schedules executed by experienced engineers for all types of prestressed and reinforced concrete structures offered to the profession. Prompt and accurate service assured. Box 4740, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SERVICES OFFERED. Experienced engineer offers prompt services to consulting engineers for the design and detailing or estimating of all types of reinforced concrete structures, including shell roofs (Midland area). Box 4742, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SERVICES OFFERED. Capacity available for consulting engineers for the design and detailing of all types of reinforced concrete and steel frame buildings, heavy industrial structures and foundations, road works, etc. Drawing Offices Services. Telephone: Tulse Hill 9195.

DESIGN SERVICES

offered to the profession by civil engineer with first-class industrial experience. Box 4743, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

FOR SALE.

FOR SALE. Steel fencing stakes, chain link, etc. E. Stephens & Son Ltd., Bath Street, London, E.C.1. Clerkenwell 1731.

SITUATION VACANT. Young civil or structural engineer (with bias towards structural steelwork) required as assistant to chief structural engineer by London consultants. Projects in U.K. and overseas. Apply in writing to Manderstam & Partners, 38 Grosvenor Gardens, London, S.W.1.

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and

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